Gratitude

In appreciation and gratitude

to The Custodian of the Two Holy Mosques

King Abdullah Bin Abdul Aziz Al Saud

And

H.R.H. Prince Sultan Bin Abdul Aziz Al Saud

Crown Prince, Deputy Premier, Minister of Defence

& Aviation and Inspector General

For their continuous support and gracious consideration,

the Saudi Building Code National Committee (SBCNC)

is honored to present the first issue of

the Saudi Building Code (SBC).

PREFACE

The Saudi Building Code (SBC) is a set of legal, administrative and technical regulations and requirements that specify the minimum standards of construction for building in order to ensure public safety and health. A Royal Decree dated 11^{th} June 2000 order the formation of a national committee composed of representatives of Saudi universities and governmental and private sectors. In September 2001, the Council of Ministers approved the general plan of the National Committee to develop a national building code for the Kingdom of Saudi Arabia.

To choose a base code for the Saudi Building Code, a number of Codes have been studied. The National Committee has been acquainted with the results of the national researches and the international codes from the U.S.A., Canada and Australia, also, the European Code, and Arab Codes. It has also sought the opinions of specialists in relevant Saudi universities, governmental and private sectors through holding a questionnaire, a symposium and specialized workshops, in the light of which, (ICC) has been chosen to be a base code for the Saudi Building Code.

The International Code Council (ICC) grants permission to the Saudi Building Code National Committee (SBCNC) to include all or any portion of material from the ICC codes, and standards in the SBC and ICC is not responsible or liable in any way to SBCNC or to any other party or entity for any modifications or changes that SBCNC makes to such documents.

Toward expanding the participation of all the specialists in the building and construction industry in the Kingdom through the governmental and private sectors, the universities and research centers, the National Committee took its own decisions related to code content by holding specialized meetings, symposiums and workshops and by the help of experts from inside and outside of Saudi Arabia.

The technical committees and sub-committees started their work in April 2003 to develop the Saudi Building Code that adapts the base code with the social and cultural environment, the natural and climatic conditions, types of soil and properties of materials in the Kingdom

The Saudi Building Code Requirements for Concrete Structures (SBC 304) were developed based on ICC code in addition to American Concrete Institute (ACI) materials. ACI grants permission to the SBCNC to include ACI materials in the SBC, and ACI is not responsible for any modifications or changes that SBCNC has made to accommodate local conditions.

The development process of SBC 304 followed the methodology approved by the Saudi Building Code National Committee. Many changes and modifications were made on ACI, such as Durability Requirements, the simplified methods for the design of two-way slab system of Appendix C, expanding some topics such as Hot Weather, taking into considerations the properties of local material such as the Saudi steel and the engineering level for those involved in the building sector.

As a follow-up to the *Saudi Building Code*, SBCNC offers a companion document, the *Saudi Building Code Steel Structural Requirements Commentary* (SBC 306C). The basic appeal of the Commentary is thus: it provides in a small package thorough coverage of many issues likely to be dealt with when using the *Saudi Building Code Steel Structural Requirements* (SBC 306) and then supplements that coverage with technical background. Reference lists, information sources and bibliographies are also included.

Strenuous effort has been made to keep the vast quantity of material accessible and its method of presentation useful. With a comprehensive yet concise summary of each section, the Commentary provides a convenient reference for regulations applicable to the construction of buildings and structures. In the chapters that follow, discussions focus on the full meaning and implications of the *Steel Structural Requirements* (SBC 306) text. Guidelines suggest the most effective method of application, and the consequences of not adhering to the SBC 306 text. Illustrations are provided to aid understanding; they do not necessarily illustrate the only methods of achieving *code* compliance.

The format of the Commentary includes the section, table and figure which is applicable to the same section in the SBC 306C. The numbers of the section, table and figure in the commentary begin with the letter R. The Commentary reflects the most up-to-date text of the 2007 *Saudi Building Code steel structural requirements* (SBC 306C). American Concrete Institute (ACI) grants permission to the SBCNC to include all or portions of ACI codes and standards in the SBC, and ACI is not responsible or liable in any way to SBCNC or to any other party or entity for any modifications or changes that SBCNC makes to such documents.

Readers should note that the Commentary (SBC 306C) is to be used in conjunction with the *Saudi Building Code steel structural requirements* (SBC 306) and not as a substitute for the code. **The Commentary is advisory only**; the code official alone possesses the authority and responsibility for interpreting the code.

Comments and recommendations are encouraged, for through your input, it can improve future editions.

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CHAPTER 1
GENERAL PROVISIONS

SECTION C1.1
SCOPE

Load and Resistance Factor Design (LRFD) is an improved approach to the design of structural steel for buildings. It involves explicit consideration of limit states, multiple load factors, and resistance factors, and implicit probabilistic determination of reliability. The designation LRFD reflects the concept of factoring both loads and resistance. This type of factoring differs from the allowable stress design (ASD) Code requirements, where only the resistance is divided by a factor of safety (to obtain allowable stress) and from the plastic design portion of that Code requirements, where only the loads are multiplied by a common load factor. The LRFD method was devised to offer the designer greater flexibility, more rationality, and possible overall economy.

SECTION C1.2
TYPES OF CONSTRUCTION

The SBC 306 emphasizes the combined importance of stiffness, strength and ductility in connection design.

An important aspect of the nominal strength of a connection, M_n, is its relationship to the strength of the connected beam $M_{p,beam}$. A connection is full strength if

$M_n > M_{p,beam}$ otherwise the connection is partial strength.

A partial strength PR connection must be designed with sufficient ductility to permit the connection components to deform and to avoid any brittle failure modes. It is also useful to define a lower limit for the strength, below which the connection can be treated as simple. Connections that transmit less than $0.2M_{p,beam}$ at a rotation of 0.02 radians can be considered to have no flexural strength for design. It should be recognized, however, that the aggregate strength of many weak partial strength connections (e.g. those with a capacity less than $0.2M_{p,beam}$) can be significant when compared to that of a few strong connections.

Connection ductility. Connection ductility is a key parameter when the deformations are concentrated in the connection elements, as is the typical case in partial strength PR connections. The ductility required will depend on the flexibility of the connections and the particular application. For example, the ductility requirement for a braced frame in a non-seismic area will generally be less than for an unbraced frame in a high seismic area.

The available ductility, θ_u, should be compared with the required rotational ductility under the full factored loads, as determined by an analysis that takes into account the nonlinear behavior of the connection. In the absence of accurate analyses of the required rotation capacity, the connection ductility may be considered adequate when the available ductility is greater than 0.03 radians. This rotation is equal to the minimum beam-to-column connection ductility as specified in the seismic provisions for special moment frames (AISC, 1997 and 1999). Many types of partial strength PR connections, such as top and seat-angle details, meet this criterion.

Connection Stiffness. Because many PR connections manifest nonlinear behavior even at low force levels, the initial stiffness of the connection, K_i, does not characterize the connection response adequately.

SECTION C1.3
MATERIALS

C1.3.1.1 **ASTM Designations.** The grades of structural steel approved for use under the LRFD Code requirements, covered by ASTM standard code requirements, extend to a yield stress of 690 MPa. Some of these ASTM standards specify a minimum yield point, while others specify a minimum yield strength. The term “yield stress” is used in the Code requirements as a generic term to denote either the yield point or the yield strength.

Properties in the direction of rolling are of principal interest in the design of steel structures. Hence, yield stress as determined by the standard tensile test is the principal mechanical property recognized in the selection of the steels approved for use under the Code requirements. It must be recognized that other mechanical and physical properties of rolled steel, such as anisotropy, ductility, notch toughness, formability, corrosion resistance, etc., may also be important to the satisfactory performance of a structure.

C1.3.1.3 Heavy Shapes

The web-to-flange intersection and the web center of heavy hot-rolled shapes as well as the interior portions of heavy plates may contain a coarser grain structure and/or lower toughness material than other areas of these products. This is probably caused by ingot segregation, as well as somewhat less deformation during hot rolling, higher finishing temperature, and a slower cooling rate after rolling for these heavy sections. This characteristic is not detrimental to suitability for service for compression members, or for non-welded members.

However, when heavy cross sections are joined by splices or connections using complete-joint-penetration welds which extend through the coarser and/or lower notch-tough interior portions, tensile strains induced by weld shrinkage may result in cracking, for example in a complete-joint-penetration welded connection of a heavy cross section beam to any column section. When members of lesser thickness are joined by complete-joint-penetration welds, which induce smaller weld shrinkage strains, to the finer grained and/or more notch-tough surface material of ASTM A6/A6M Group 4 and 5 shapes and heavy built-up cross sections, the potential for cracking is significantly lower, for example in a complete-joint-penetration groove welded connection of a non-heavy cross-section beam to a heavy cross-section column. For critical applications such as primary tension members, material should be specified to provide adequate toughness at service temperatures. Because of differences in the strain rate between the Charpy V-notch (CVN) impact test and the strain rate experienced in actual structures, the CVN test is conducted at a temperature higher than the anticipated service temperature for the structure. The location of the CVN test is shown in Figure C1.3-1.

The toughness requirements of Section 1.3.1.3 are intended only to provide material of reasonable toughness for ordinary service applications. For unusual applications

and/or low temperature service, more restrictive requirements and/or toughness requirements for other section sizes and thicknesses may be appropriate. To minimize the potential for fracture, the notch toughness requirements of Section 1.3.1.3 must be used in conjunction with good design and fabrication procedures. Specific requirements are given in Sections 10.1.5, 10.1.6, 10.2.6, 10.2.8, and 13.2.2.

C1.3.3 **Bolts, Washers, and Nuts**. The ASTM standard for A307 bolts covers two grades of fasteners. Either grade may be used under the LRFD Code requirements; however, it should be noted that Gr. B is intended for pipe flange bolting and Gr. A is the grade long in use for structural applications.

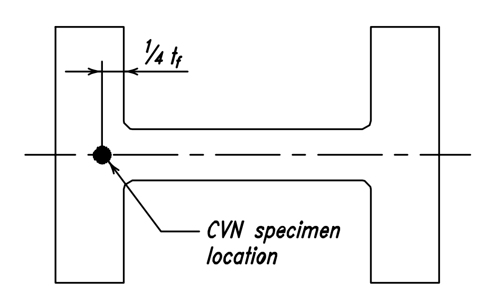


Figure. C1.3-1 Location from which Charpy impact specimen shall be taken.

C1.3.4 **Anchor Rods and Threaded Rods.** Since there is a limit on the maximum available length of A325 or A325M and A490 or A490M bolts, the attempted use of these bolts for anchor rods with design lengths longer than the maximum available lengths has presented problems in the past. The inclusion of A449 and A354 materials in these Code requirements allows the use of higher strength material for bolts longer than A325 or A325M and A490 or A490M bolts. The designer should be aware that pretensioning of anchor rods is not recommended due to relaxation and the potential for stress corrosion after pretensioning.

The designer should specify the appropriate thread and SAE fit for threaded rods used as load-carrying members.

C1.3.5 **Filler Metal and Flux for Welding**. The filler metal code requirements issued by the American Welding Society (AWS) are general code requirements which include filler metals suitable for building construction, as well as consumables that would not be suitable for building construction. For example, some electrodes covered by the code requirements are specifically limited to single pass applications, while others are restricted to sheet metal applications. Many of the filler metals listed are “low hydrogen,” that is, they deposit filler metal with low levels of diffusible hydrogen. Other materials are not. Filler metals listed under the various AWS A5 code requirements may or may not have required impact toughness, depending on the specific electrode classification. Section 10.2.6 has identified certain welded joints where notch toughness of filler metal is needed in building construction. However, on structures subject to dynamic loading, filler

metals may be required to deliver notch-tough weld deposits in other joints. Filler metals may be classified in either the as-welded or post weld heat-treated (stress-relieved) condition. Since most structural applications will not involve stress relief, it is important to utilize filler materials that are classified in conditions similar to those experienced by the actual structure.

When specifying filler metal and/or flux by AWS designation, the applicable standard code requirements should be carefully reviewed to assure a complete understanding of the designation reference. This is necessary because the AWS designation systems are not consistent. For example, in the case of electrodes for shielded metal arc welding (AWS A5.1), the first two or three digits indicate the nominal tensile strength classification, in ksi, of the filler metal and the final two digits indicate the type of coating. For metric designations, the first two digits times 10 indicate the nominal tensile strength classification in MPa. In the case of mild steel electrodes for submerged arc welding (AWS A5.17), the first one or two digits times 10 indicate the nominal tensile strength classification for both U.S. Customary and metric units, while the final digit or digits times 10 indicate the testing temperature in degrees Celsius, for filler metal impact tests. In the case of low-alloy steel covered arc welding electrodes (AWS A5.5), certain portions of the designation indicate a requirement for stress relief, while others indicate no stress relief requirement.

Engineers do not, in general, specify the exact filler metal to be employed on a particular structure. Rather, the decision as to which welding process and which filler metal is to be utilized, is usually left with the fabricator or erector. To ensure that the proper filler metals are used, codes restrict the usage of certain filler materials, or impose qualification testing to prove the suitability of the specific electrode.

SECTION C1.4
LOADS AND LOAD COMBINATIONS

The load factors and load combinations are developed based on the recommended minimum loads given in SBC-301.

The load factors and load combinations recognize that when several loads act in combination with the dead load (e.g., dead plus live plus wind), only one of these takes on its maximum lifetime value, while the other load is at its "arbitrary point-in-time value" (i.e., at a value which can be expected to be on the structure at any time). For example, under dead, live, and wind loads the following combinations are appropriate:

$$\gamma_D D + \gamma_L L \qquad \text{(C1.4-1)}$$

$$\gamma_D D + \gamma_{La} L_a + \gamma_W W \qquad \text{(C1.4-2)}$$

$$\gamma_D D + \gamma_L L + \gamma_{W_a} W_a \qquad \text{(C1.4-3)}$$

Where γ is the appropriate load factor as designated by the subscript symbol. Subscript, “a” refers to an “arbitrary point-in-time” value.

The mean value of arbitrary point-in-time live load L_a is on the order of 0.24 to 0.4 times the mean maximum lifetime live load L for many occupancies, but its dispersion is far greater. The arbitrary point-in-time wind load W_a, acting in conjunction with the maximum lifetime live load, is the maximum daily wind. It turns out that $\gamma_{W_a} W_a$ is a negligible quantity so only two load combinations remain:

$$1.2D + 1.6L \qquad \text{(C1.4-4)}$$

$$1.2D + 0.5L + 1.3W \qquad \text{(C1.4-5)}$$

The load factor 0.5 assigned to L in the second formula reflects the statistical properties of L_a, but to avoid having to calculate yet another load, it is reduced so it can be combined with the maximum lifetime wind load.

SECTION C1.5
DESIGN BASIS

C1.5.1 **Required Strength at Factored Loads**. LRFD permits the use of both elastic and plastic structural analyses. LRFD provisions result in essentially the same methodology for, and end product of, plastic design except that the LRFD provisions tend to be slightly more liberal, reflecting added experience and the results of further research.

In some circumstances, as in the proportioning of the bracing members that carry no calculated forces (see Section 3.3) and of connection components (see Item 10.1.7), the required strength is explicitly stated in the Code requirements.

C1.5.2 **Limit States.** A limit state is a condition which represents the limit of structural usefulness. Limit states may be dictated by functional requirements, such as maximum deflections or drift; they may be conceptual, such as plastic hinge or mechanism formation; or they may represent the actual collapse of the whole or part of the structure, such as fracture or instability. Design criteria ensure that a limit state is violated only with an acceptably small probability by selecting the combination of load and resistance factors and nominal load and resistance values which will never be exceeded under the design assumptions.

Two kinds of limit states apply for structures: limit states of strength which define safety against extreme loads during the intended life of the structure, and limit states of serviceability which define functional requirements. The LRFD Code requirements, like other structural codes, focuses on the limit states of strength because of overriding considerations of public safety for the life, limb, and property of human beings. This does not mean that limit states of serviceability are not important to the designer, who must equally ensure functional performance and

economy of design. However, these latter considerations permit more exercise of judgment on the part of designers. Minimum considerations of public safety, on the other hand, are not matters of individual judgment and, therefore, code requirements dwell more on the limit states of strength than on the limit states of serviceability.

Limit states of strength vary from member to member, and several limit states may apply to a given member. The following limit states of strength are the most common: onset of yielding, formation of a plastic hinge, formation of a plastic mechanism, overall frame or member instability, lateral-torsional buckling, local buckling, tensile fracture, development of fatigue cracks, deflection instability, alternating plasticity, and excessive deformation. The most common serviceability limit states include unacceptable elastic deflections and drift, unacceptable vibrations, and permanent deformations.

C1.5.3 **Design for Strength**. The general format of the LRFD Code requirements is given by the formula:

$$\Sigma\gamma_i Q_i \leq \phi R_n \qquad (C1.5\text{-}1)$$

where

Σ = summation

i = type of load, i.e., dead load, live load, wind, etc.

Q_i = nominal load effect

γ_i = load factor corresponding to Q_i

$\Sigma\gamma_i Q_i$ = required strength

R_n = nominal strength

ϕ = resistance factor corresponding to R_n

ϕR_n = design strength

The left side of Equation *(C1.5-1)* represents the required resistance computed by structural analysis based upon assumed loads, and the right side of Equation *(C1.5-1)* represents a limiting structural capacity provided by the selected members. In LRFD, the designer compares the effect of factored loads to the strength actually provided. The term design strength refers to the resistance or strength ϕR_n that must be provided by the selected member. The load factors γ and the resistance factors reflect the fact that loads, load effects (the computed forces and moments in the structural elements), and the resistances can be determined only to imperfect degrees of accuracy. The resistance factor ϕ is equal to or less than 1.0 because there is always a chance for the actual resistance to be less than the nominal value R_n computed by the equations given in Chapters 4 through 11. Similarly, the load factors γ reflect the fact that the actual load effects may deviate from the nominal

values of Q_i computed from the specified nominal loads. These factors account for unavoidable inaccuracies in the theory, variations in the material properties and dimensions, and uncertainties in the determination of loads. They provide a margin of reliability to account for unexpected loads. They do not account for gross error or negligence. The LRFD Code requirements is based on (1) probabilistic models of loads and resistance, (2) a calibration of the LRFD criteria to the ASD Code requirements for selected members, and (3) the evaluation of the resulting criteria by judgment and past experience aided by comparative design office studies of representative structures.

C1.5.4 **Design for Serviceability and Other Considerations**: Nominally, serviceability should be checked at the unfactored loads. For combinations of gravity and wind or seismic loads some additional reduction factor may be warranted.

CHAPTER 2
DESIGN REQUIREMENTS

SECTION C2.5
LOCAL BUCKLING

For the purposes of these Code requirements, steel sections are divided into compact sections, non-compact sections, and sections with slender compression elements. Compact sections are capable of developing a fully plastic stress distribution and they possess a rotational capacity of approximately 3 before the onset of local buckling (Yura, Galambos, and Ravindra, 1978). Non-compact sections can develop the yield stress in compression elements before local buckling occurs, but will not resist inelastic local buckling at the strain levels required for a fully plastic stress distribution. Slender compression elements buckle elastically before the yield stress is achieved.

The dividing line between compact and non-compact sections is the limiting width-thickness ratio λ_p. For a section to be compact, all of its compression elements must have width-thickness ratios equal to or smaller than the limiting λ_p.

TABLE C2.5-1
Limiting Width-Thickness Ratios for Compression Elements

Description of Element	Width-Thickness Ratio	Limiting Width-thickness Ratios λ_p Non-seismic	Limiting Width-thickness Ratios λ_p Seismic
Flanges of I-shaped sections (including hybrid sections) and channels in flexure [a]	b/t	$0.38\sqrt{E/F_y}$	$0.31\sqrt{E/F_y}$
Webs in combined flexural and axial compression	h/t_w	For $P_u/\phi_b P_y \le 0.125$	
		$3.76\sqrt{\frac{E}{F_y}}\left(1-\frac{2.75P_u}{\phi_b P_y}\right)$	$3.05\sqrt{\frac{E}{F_y}}\left(1-\frac{1.54P_u}{\phi_b P_y}\right)$
		For $P_u/\phi_b P_y > 0.125$	
		$1.12\sqrt{\frac{E}{F_y}}\left(2.33-\frac{P_u}{\phi_b P_y}\right) \ge 1.49\sqrt{\frac{E}{F_y}}$	

[a] For hybrid beams use F_{yf} in place of F_y

A greater inelastic rotation capacity than provided by the limiting values λ_p given in Table C2.5-1 may be required for some structures in areas of high seismicity. It has been suggested that in order to develop a ductility of from 3 to 5 in a structural member, ductility factors for elements would have to lie in the range of 5 to 15. Thus, in this case it is prudent to provide for an inelastic rotation of 7 to 9 times the elastic rotation (Chopra and Newmark, 1980). In order to provide for this rotation capacity, the limits λ_p for local flange and web buckling would be as shown in Table C2.5-1 (Galambos, 1976).

More information on seismic design is contained in the AISC Seismic Provisions for Structural Steel Buildings (AISC, 1997) and the Seismic Provisions for Structural Steel Buildings Supplement No. 1 (AISC, 1999).

Another limiting width-thickness ratio is λ_r, representing the distinction between non-compact sections and sections with slender compression elements. As long as the width-thickness ratio of a compression element does not exceed the limiting value λ_r, local elastic buckling will not govern its strength. However, for those cases where the width-thickness ratios exceed λ_r, elastic buckling strength must be considered. A design procedure for such slender-element compression sections, based on elastic buckling of plates, is given in Section 2.5.3. The effective width Equation 2.5-12 applies strictly to stiffened elements under uniform compression. It does not apply to cases where the compression element is under stress gradient. A method of dealing with the stress gradient in a compression element is provided in Section 2.2 of the AISI Code requirements for the Design of Cold-Formed Steel Structural Members (1996). Exceptions are girders with slender webs. Such plate girders are capable of developing post-buckling strength in excess of the elastic buckling load. A design procedure for plate girders including tension field action is given in Section 7.

The values of the limiting ratios λ_p and λ_r specified in Table 2.5-1 are similar to those in AISC (1989) and Table 2.3.3.3 of Galambos (1976), except that: (1) λ_p = $0.38\sqrt{E/F_y}$, limited in Galambos (1976) to indeterminate beams when moments are determined by elastic analysis and to determinate beams, was adopted for all conditions on the basis of Yura et al. (1978); and (2) $\lambda_p = 0.045E/F_y$, for plastic design of circular hollow sections was obtained from Sherman (1976).

The high shape factor for circular hollow sections makes it impractical to use the same slenderness limits to define the regions of behavior for different types of loading. In Table 2.5-1, the values of λ_p for a compact shape that can achieve the plastic moment, and λ_r for bending, are based on an analysis of test data from several projects involving the bending of pipes in a region of constant moment (Sherman and Tanavde, 1984, and Galambos, 1998). The same analysis produced the equation for the inelastic moment capacity in Table 6.1-1 in Section 6.1. However, a more restrictive value of λ_p is required to prevent inelastic local buckling from limiting the plastic hinge rotation capacity needed to develop a mechanism in a circular hollow beam section (Sherman, 1976).

The values of λ_r for axial compression and for bending are both based on test data. The former value has been used in building code requirements since 1968 (Winter, 1970). Sections 2.5 and 6.1 also limit the diameter-to-thickness ratio for any circular section to $0.45E/F_y$. Beyond this, the local buckling strength decreases rapidly, making it impractical to use these sections in building construction.

Following the SSRC recommendations (Galambos, 1998) and the approach used for other shapes with slender compression elements, a Q factor is used for circular sections to account for interaction between local and column buckling. The Q factor is the ratio between the local buckling stress and the yield stress. The local buckling stress for the circular section is taken from the inelastic AISI criteria (Winter, 1970) and is based on tests conducted on fabricated and manufactured cylinders. Subsequent tests on fabricated cylinders (Galambos, 1998) confirm that this equation is conservative.

The definitions of the width and thickness of compression elements agree with the 1978 AISC ASD Code requirements with minor modifications. Their

applicability extends to sections formed by bending and to unsymmetrical and hybrid sections.

For built-up I-shaped sections under axial compression, modifications have been made to the flange local buckling criterion to include web-flange interaction. The k_c in the λ_r limit, in Equations 2.5-7 and 2.5-8 and the elastic buckling Equation 2.5-8 are the same that are used for flexural members. Theory indicates that the web-flange interaction in axial compression is at least as severe as in flexure. Rolled shapes are excluded from this criterion because there are no standard sections with proportions where the interaction would occur. In built-up sections where the interaction causes a reduction in the flange local buckling strength, it is likely that the web is also a thin stiffened element.

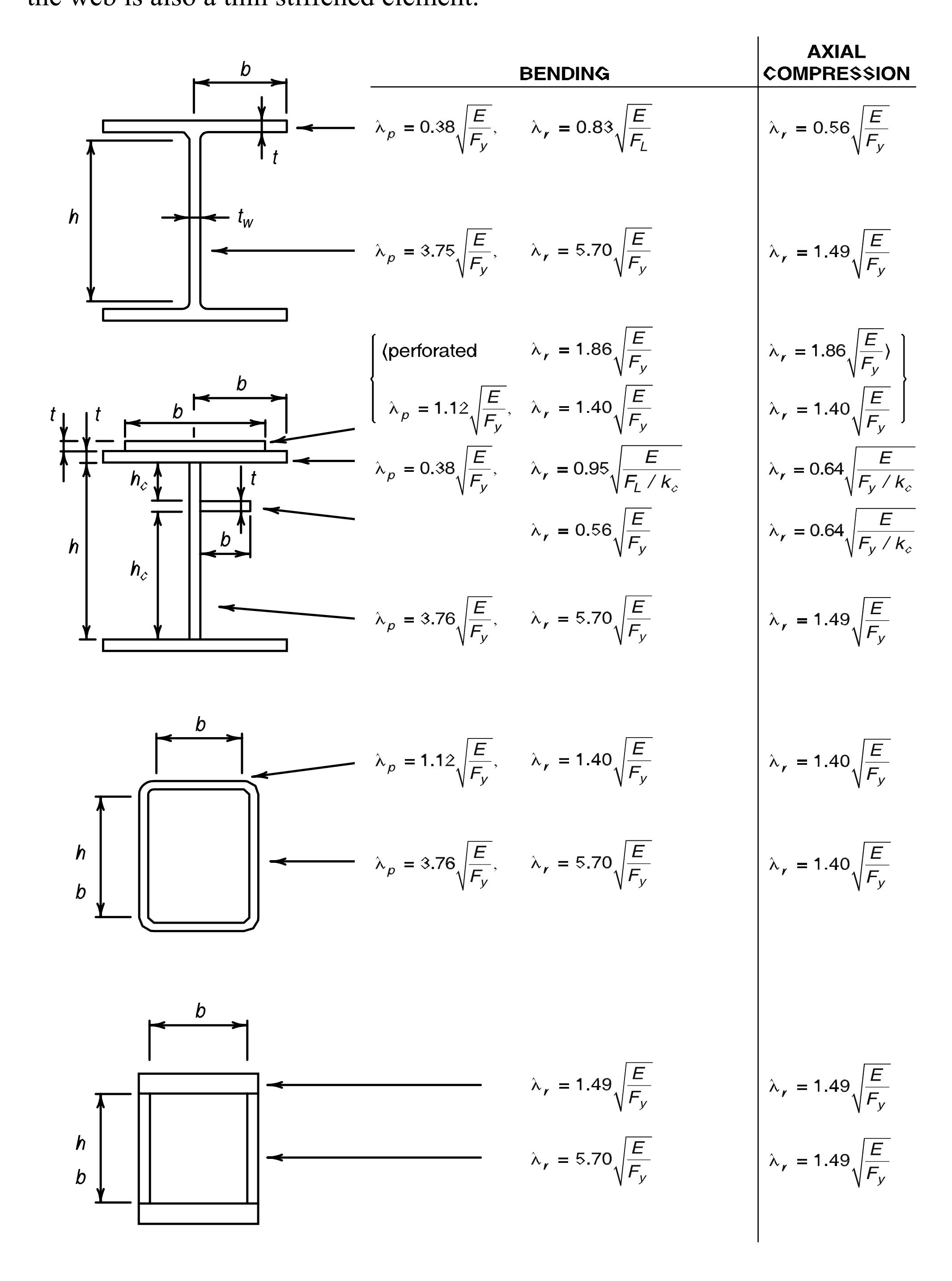


Figure. C2.5-1 Selected examples of Table 2.5-1 requirements.

The k_c factor accounts for the interaction of flange and web local buckling demonstrated in experiments conducted by Johnson (1985). The maximum limit of 0.763 corresponds to $F_{cr} = 0.69E / \lambda^2$ which was used as the local buckling strength in earlier editions of both the ASD and LRFD Code requirements. An $h/t_w = 27.5$ is required to reach $k_c = 0.763$. Fully fixed restraint for an unstiffened compression element corresponds to $k_c = 1.3$ while zero restraint gives $k_c = 0.42$. Because of web-flange interactions it is possible to get $k_c < 0.42$ from the new k_c formula. If $h/t_w > 5.70\sqrt{E/F_y}$ use $h/t_w = 5.70\sqrt{E/F_y}$ in the k_c equation, which corresponds to the 0.35 limit.

Illustrations of some of the requirements of Table 2.5-1 of SBC 306 are shown in Figure C2.5-1.

SECTION C2.7
LIMITING SLENDERNESS RATIOS

Chapters 4 and 5 provide reliable criteria for resistance of axially loaded members based on theory and confirmed by tests for all significant parameters including slenderness. The advisory upper limits on slenderness contained in Section 2.7 are based on professional judgment and practical considerations of economics, ease of handling, and care required to minimize inadvertent damage during fabrication, transport, and erection. Out-of-straightness within reasonable tolerances does not affect the strength of tension members, and the effect of out-of-straightness within specified tolerances on the strength of compression members is accounted for in formulas for resistance. Applied tension tends to reduce, whereas compression tends to amplify, out-of-straightness. Therefore, more liberal criteria are suggested for tension members, including those subject to small compressive forces resulting from transient loads such as earthquake and wind. For members with slenderness ratios greater than 200, these compressive forces correspond to $\phi_c F_{cr}$ less than 18 MPa.

CHAPTER 3
FRAMES AND OTHER STRUCTURES

SECTION C3.1
SECOND ORDER EFFECTS

For frames under combined gravity and lateral loads, drift Δ (horizontal deflection caused by applied loads) occurs at the start of loading. In un-braced frames, additional secondary bending moments, known as the $P\Delta$ moments, may be developed in the columns and beams of the lateral load-resisting systems in each story. P is the total gravity load above the story and Δ is the story drift. As the applied load increases, the $P\Delta$ moments also increase. Therefore, the $P\Delta$ effect must often be accounted for in frame design. Similarly, in braced frames, increases in axial forces occur in the members of the bracing systems; however, such effects are usually less significant. The designer should consider these effects for all types of frames and determine if they are significant. Since $P\Delta$ effects can cause frame drifts to be larger than those calculated by ignoring them, they should also be included in the service load drift analysis when they are significant.

In un-braced frames designed by plastic analysis, the limit of $0.75\phi_c P_y$ on column axial loads has been retained to help ensure stability.

The designer may use second-order elastic analysis to compute the maximum factored forces and moments in a member. These represent the required strength. Alternatively, for structures designed on the basis of elastic analysis, the designer may use first order analysis and the amplification factors B_1 *and* B_2.

In the general case, a member may have first order moments not associated with sidesway which are multiplied by B_1, and first order moments produced by forces causing sidesway which are multiplied by B_2.

The factor B_2 applies only to moments caused by forces producing sidesway and is calculated for an entire story. In building frames designed to limit Δ_{oh}/L to a predetermined value, the factor B_2 may be found in advance of designing individual members.

Drift limits may also be set for design of various categories of buildings so that the effect of secondary bending can be insignificant. It is conservative to use the B_2 factor with the sum of the sway and the no-sway moments, i.e., with $M_{lt} + M_{nt}$.

The two kinds of first order moment M_{nt} and M_{lt} may both occur in sidesway frames from gravity loads. M_{nt} is defined as a moment developed in a member with frame sidesway prevented. If a significant restraining force is necessary to prevent sidesway of an unsymmetrical structure (or an un-symmetrically loaded symmetrical structure), the moments induced by releasing the restraining force will be M_{lt} moments, to be multiplied by B_2. In most reasonably symmetric frames, this effect will be small. If such a moment B_2M_{lt} is added algebraically to the B_1M_{nt} moment developed with sidesway prevented, a fairly accurate value of M_u will result. End moments produced in sidesway frames by lateral loads from wind or earthquake will always be M_{lt} moments to be multiplied by B_2.

When first order end moments in members subjected to axial compression are magnified by B_1 and B_2 factors, equilibrium requires that they be balanced by moments in connected members. Connections shall also be designed to resist the magnified end moments.

For beam columns with transverse loadings, the second-order moment can be approximated by using the following equation

$$C_m = 1 + \psi P_u / P_{e1} \qquad \text{(C3.1-1)}$$

for simply supported members

where

ψ = $\dfrac{\pi^2 \delta_o EI}{M_o L^2} - 1$

δ_o = maximum deflection due to transverse loading, mm

M_o = maximum factored design moment between supports due to transverse loading, N-mm

For restrained ends, some limiting cases are given in Table C3.1-1 together with two cases of simply supported beam-columns. These values of C_m are always used with the maximum moment in the member. For the restrained-end cases, the values of B_1 will be most accurate if values of $K < 1.0$ corresponding to the end boundary conditions are used in calculating P_{e1}. In lieu of using the equations above, $C_m = 1.0$ can be used conservatively for transversely loaded members with unrestrained ends and 0.85 for restrained ends.

If, as in the case of a derrick boom, a beam-column is subject to transverse (gravity) load and a calculable amount of end moment, the value δ_0 should include the deflection between supports produced by this moment.

Stiffness reduction adjustment due to column inelasticity is permitted.

TABLE C3.1-1
Amplification Factors for ψ and C_m

Case	Ψ	C_m
P_u	0	1.0
	−0.4	$1-0.4\dfrac{P_u}{P_{e1}}$
	−0.4	$1-0.4\dfrac{P_u}{P_{e1}}$
	−0.2	$1-0.2\dfrac{P_u}{P_{e1}}$
L/2	−0.3	$1-0.3\dfrac{P_u}{P_{e1}}$
	−0.2	$1-0.2\dfrac{P_u}{P_{e1}}$

SECTION C3.2
FRAME STABILITY

The stability of structures must be considered from the standpoint of the structure as a whole, including not only the compression members, but also the beams, bracing system, and connections (Galambos, 1998). The stability of individual elements must also be provided.

The effective length concept is one method of estimating the interaction effects of the total frame on a compression element being considered. This concept uses K factors to equate the strength of a framed compression element of length L to an equivalent pin-ended member of length KL subject to axial load only. Other rational methods are available for evaluating the stability of frames subject to gravity and side loading and individual compression members subject to axial load and moments.

The ratio K, effective column length to actual unbraced length, may be greater or less than 1.0, depending upon if the column is part of an unbraced frame or braced frame. The theoretical K values for six idealized conditions in which joint rotation and translation are either fully realized or nonexistent are tabulated in Table C3.2-1.

Also shown are suggested design values recommended by the Structural Stability Research Council (SSRC) for use when these conditions are approximated in actual design. In general, these suggested values are slightly higher than their theoretical equivalents, since joint fixity is seldom fully realized.

TABLE C3.2-1
K values for Columns

Buckled shape of column is shown by dashed line.	(a)	(b)	(c)	(d)	(e)	(f)
Theoretical K value	0.5	0.7	1.0	1.0	2.0	2.0
Recommended design value when ideal conditions are approximated	0.65	0.80	1.2	1.0	2.10	2.0
End condition code	Rotation fixed and translation fixed Rotation free and translation fixed Rotation fixed and translation free Rotation free and translation free					

While in some cases masonry walls provide enough lateral support for building frames to control lateral deflection, light curtain wall construction and wide column spacing can create a situation where only the bending stiffness of the frame provides this support. In this case the effective length factor K for an unbraced length of column L is dependent upon the bending stiffness provided by the other in-plane members entering the joint at each end of the unbraced segment. If the combined stiffness provided by the beams is sufficiently small, relative to that of the unbraced column segments, KL could exceed two or more story heights.

Translation of the joints in the plane of a truss is inhibited and, due to end restraint, the effective length of compression members might be assumed to be less than the distance between panel points. However, it is usual practice to take K as equal to 1.0. If all members of the truss reached their ultimate load capacity simultaneously, the restraints at the ends of the compression members would be greatly reduced.

Once a trial selection of framing members has been made, the use of the alignment chart in Figures C3.2-1a and b affords a fairly rapid method for determining adequate *K* values. However, it should be noted that this alignment chart is based upon assumptions of idealized conditions which seldom exist in real structures (ASCE Task Committee on Effective Length, 1997).

The alignment chart for sidesway uninhibited shown in Figure C3.2-1b is based on the following equation:

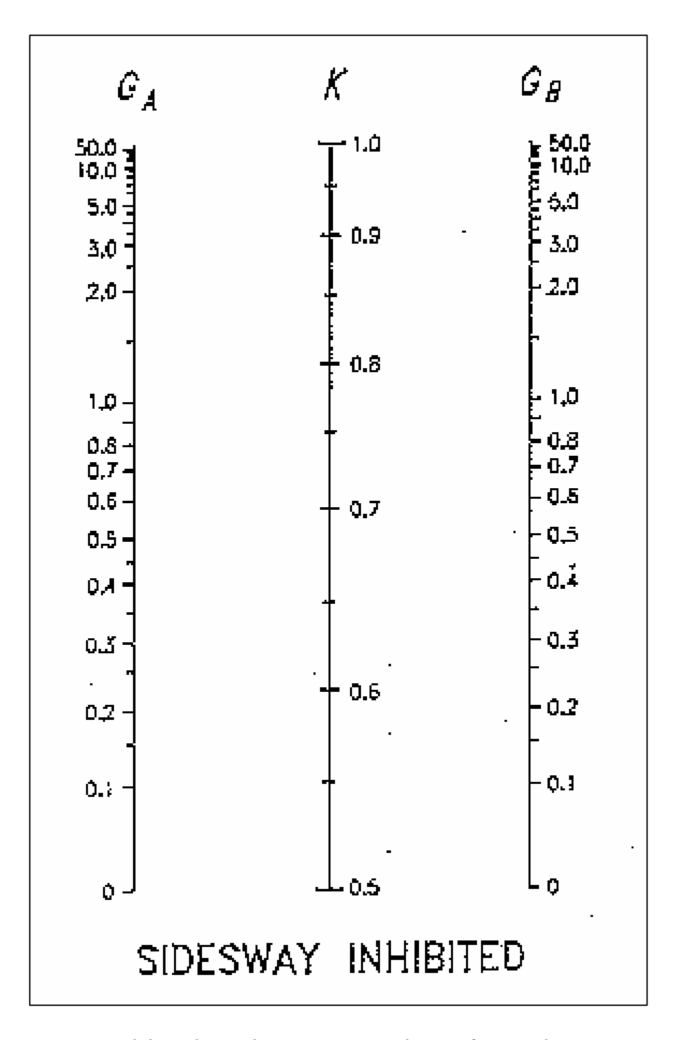
$$\frac{G_A G_B (\pi / K)^2 - 36}{6(G_A + G_B)} - \frac{(\pi / K)}{\tan(\pi / K)} = 0 \qquad \text{(C3.2-1)}$$

with G defined as

$$G = \frac{\Sigma(EI / L)_c}{\Sigma(EI / L)_g} \qquad \text{(C3.2-2)}$$

The expression for G given in the footnote of the alignment chart has assumed that E of the beams and columns are the same. However, the alignment chart is valid for different materials if Equation C3.2-2 is used. An equation for the sidesway-inhibited chart can be found in ASCE Task Committee on Effective Length (1997).

The theoretical K-factors that are less than 1.0 (Cases (a) and (b) in Table C3.2-1 and the sidesway inhibited alignment chart in Figure C3.2-1a, are based on the assumption that there is no relative lateral movement of the ends of the column. When bracing is proportioned by the requirements of Section 3.3, *K* equal to 1.0 should be used, not values less than 1.0, because a small relative movement of the brace points is anticipated.



Notes for Figure C3.2-1a and b: The subscripts A and B refer to the joints at the two ends of the column section being considered. G is defined as

$$G = \frac{\Sigma(I_c / L_c)}{\Sigma(I_g / L_g)}$$

in which Σ indicates a summation of all members rigidly connected to that joint and lying on the plane in which buckling of the column is being considered. I_c *is the moment of inertia and* L_c *the unsupported length of a column section, and* I_g *is the moment of inertia and* L_g *the unsupported length of a girder or other restraining member.* I_c *and* I_g *are taken about axes perpendicular to the plane of buckling being considered.*

For column ends supported by but not rigidly connected to a footing or foundation, G is theoretically infinity, but, unless actually designed as a true friction-free pin, may be taken as "10" for practical designs. If the column end is rigidly attached to a properly designed footing, G may be taken as zero.

Figure. C3.2-1(a) Alignment chart for effective length of columns in continuous frames – Sidesway Inhibited.

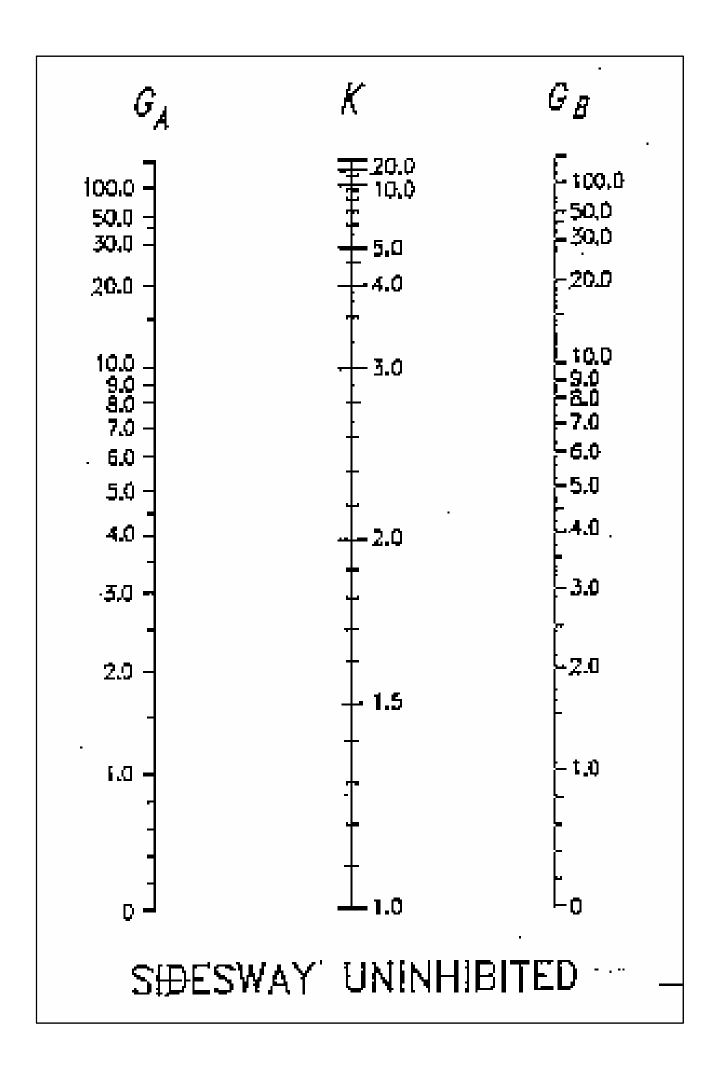


Figure. C3.2-1(b) Alignment chart for effective length of columns in continuous frames – Sidesway Uninhibited.

SECTION C3.3 STABILITY BRACING

C3.3.1 **Scope.** The design requirements consider two general types of bracing systems, relative and nodal, as shown in Figure C3.3-1 A relative column brace system (such as diagonal bracing or shear walls) is attached to two locations along the length of the column that defines the unbraced length. The relative brace system shown consists of the diagonal and the strut that controls the movement at one end of the unbraced length, A, with respect to the other end of the unbraced length, *B*. The diagonal and the strut both contribute to the strength and stiffness of the relative brace system. However, when the strut is a floor beam, its stiffness is large compared to the diagonal so the diagonal controls the strength and stiffness of the relative brace. A nodal brace controls the movement only at the particular brace point, without direct interaction with adjacent braced points. Therefore, to define an unbraced length, there must be additional adjacent brace points as shown in Figure C3.3-1. The two nodal column braces at *C* and *D* that are attached to the rigid abutment define the unbraced length for which K = 1.0 can be used. For beams, a cross frame between two adjacent beams at mid-span is a nodal brace because it prevents twist of the beams only at the particular cross frame location. The unbraced length is half the span length. The twist at the ends of the two beams is prevented by the beam-to-column connections at the end

supports. Similarly, a nodal lateral brace attached at mid-span to the top flange of the beams and a rigid support assumes that there is no lateral movement at the column locations.

The brace requirements will enable a member to potentially reach a maximum load based on the unbraced length between the brace points and $K = 1.0$.

Winter (1958 and 1960) developed the concept of dual criterion for bracing design, strength and stiffness. The brace force is a function of the initial column out-of-straightness, Δ_o, and the brace stiffness, β. For a relative brace system, the relationship between column load, brace stiffness and sway displacement is shown in Figure C3.3-2. If $\beta = \beta_i$, the critical brace stiffness for a perfectly plumb member, then $P = P_e$ only if the sway deflection gets very large. Unfortunately, such large displacements produce large brace forces. For practical design, Δ must be kept small at the factored load level.

The brace stiffness requirements, β_{br}, for frames, columns, and beams were chosen as twice the critical stiffness. The $\phi = 0.75$ specified for all brace stiffness requirements is consistent with the implied resistance factor for elastic Euler column buckling, i.e. $0.877 \times \phi_c = 0.75$. For the relative brace system shown in Figure C3.3-2, $\beta_{br} = 2\beta_i$ gives $P_{br} = 0.4\%\ P_e$ for $\Delta_o = 0.002L$. If the brace stiffness provided, β_{act}, is different from the requirement, then the brace force or brace moment can be multiplied by the following factor:

$$\frac{1}{2 - \dfrac{\beta_{br}}{\beta_{act}}} \qquad \text{(C3.3-1)}$$

No ϕ is specified in the brace strength requirements since ϕ is included in the component design strength provisions in other chapters of this Code requirements.

The initial displacement, Δ_o, for relative and nodal braces is defined with respect to the distance between adjacent braces. The initial Δ_o is a displacement from the straight position at the brace points caused by sources other than brace elongations from gravity loads or compressive forces, such as displacements caused by wind or other lateral forces, erection tolerances, column shortening, etc. The brace force recommendations for frames, columns and beam lateral bracing are based on an assumed $\Delta_o = 0.002L$, where L is the distance between adjacent brace points. For torsional bracing of beams, an initial twist angle, θ_o, is assumed where $\theta_o = 0.002L/h_o$, and h_o is the distance between flange centroids. For other Δ_o and θ_o values, use direct proportion to modify the brace strength requirements, P_{br} and M_{br}. For cases where it is unlikely that all columns in a story are out-of-plumb in the same direction, Chen and Tong (1994) recommend an average $\Delta_o = 0.002L/\sqrt{n_o}$ where n_o columns, each with a random Δ_o, are to be stabilized by the brace system. This reduced Δ_o would be appropriate when combining the stability brace forces with wind and seismic forces.

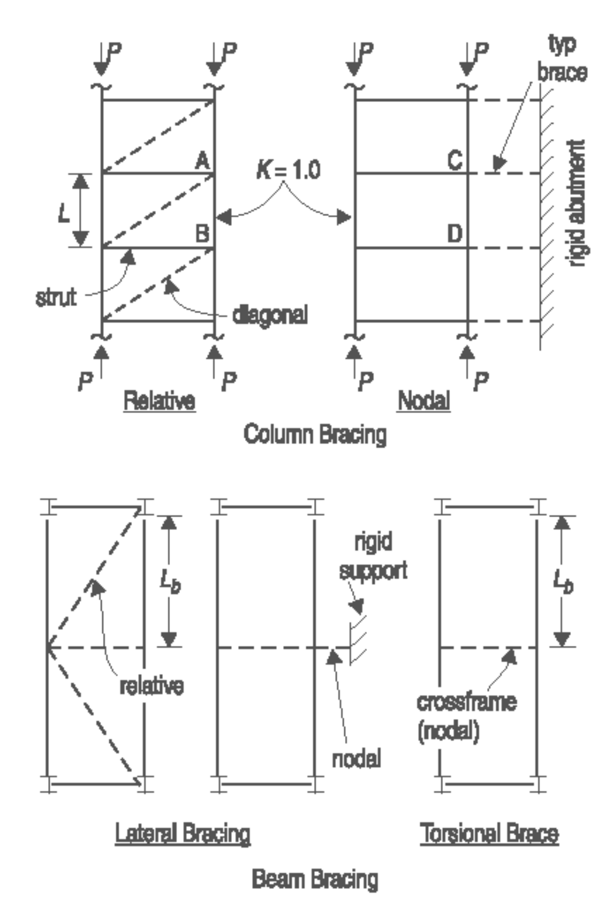


Figure. C3.3-1. Types of bracing

Brace connections, if they are flexible or can slip, should be considered in the evaluation of the bracing stiffness as follows:

$$\frac{1}{\beta_{act}} = \frac{1}{\beta_{conn}} + \frac{1}{\beta_{brace}} \tag{C3.3-2}$$

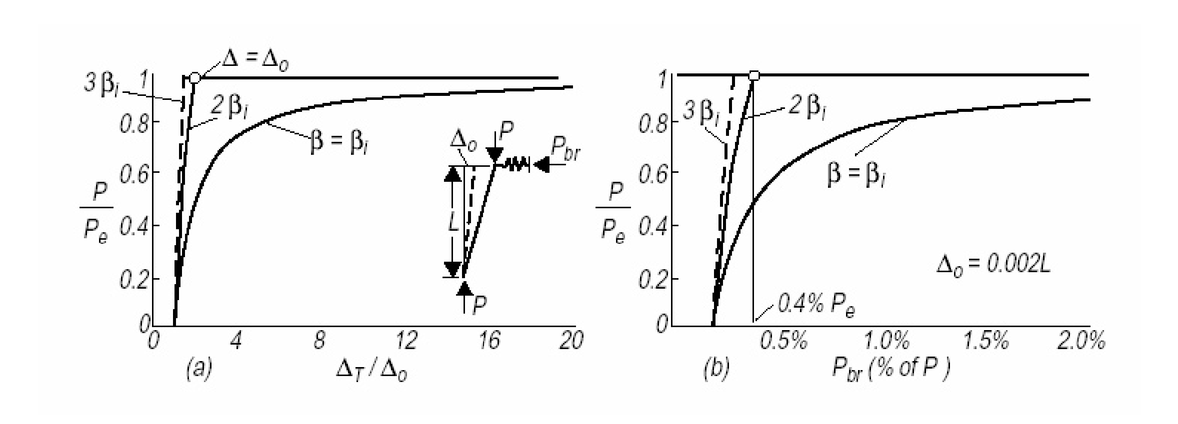


Figure. C3.3-2. Effect of initial out-of-plumbness.

The brace system stiffness, β_{act}, is less than the smaller of the connection stiffness, β_{conn}, or the stiffness of the brace, β_{brace}. Slip in connections with standard holes need not be considered except when only a few bolts are used. When evaluating the bracing of rows of columns or beams, consideration must be given to the accumulation of the brace forces along the length of the brace that results in a different displacement at each beam or column location. In general, brace forces can be minimized by increasing the number of braced bays and using stiff braces.

C3.3.3 **Columns.** For nodal column bracing, the critical stiffness is a function of the number of intermediate braces (Winter, 1958 and 1960). For one intermediate brace, $\beta_i = 2P/L_b$, and for many braces $\beta_i = 4P/L_b$. The relationship between the critical stiffness and the number of braces, n, can be approximated (Yura, 1995) as $\beta_i = N_iP/L_b$, where $N_i = 4 - 2/n$. The most severe case (many braces) was adopted for the brace stiffness requirement $\beta_{br} = 2 \times 4P / L_b$. The brace stiffness, Equation C3.3-6, can be reduced by the ratio, $N_i/4$, to account for the actual number of braces.

The unbraced length, L_b, in Equations C3.3-4 and C3.3-6 is assumed to be equal to the length L_q that enables the column to reach P_u. When the actual bracing spacing is less than L_q, the calculated required stiffness may become quite conservative since the stiffness equations are inversely proportional to L_b. In such cases, L_q can be substituted for L_b.

Winter's rigid model would derive a brace force of 0.8 percent P_u which accounts only for lateral displacement force effects. To account for the additional force due to member curvature, this theoretical force has been increased to one percent P_u.

C3.3.4 **Beams.** Beam bracing must prevent twist of the section, not lateral displacement. Both lateral bracing (for example, joists attached to the compression flange of a simply supported beam) and torsional bracing (for example, a cross frame or diaphragm between adjacent girders) can effectively control twist. Lateral bracing systems that are attached near the beam centroid are ineffective. For beams with double curvature, the inflection point cannot be considered a brace point because twist occurs at that point (Galambos, 1998). A lateral brace on one flange near the inflection point also is ineffective. In double curvature cases, the lateral brace near the inflection point must be attached to both flanges to prevent twist, or torsional bracing must be used. The beam brace requirements are based on the recommendations by Yura (1993).

C3.3.4.1 **Lateral Bracing.** For lateral bracing, the following stiffness requirement was derived following Winter's approach:

$$\beta_{br} = 2N_i\,(C_bP_f\,)\,C_tC_d\,/\,\phi L_b \qquad \text{(C3.3-3)}$$

where

N_i = 1.0 for relative bracing

= $(4-2/n)$ for discrete bracing

n = number of intermediate braces

P_f = beam compressive flange force

= $\pi^2 EI_{yc}/L_b^2$

I_{yc} = out-of-plane moment of inertia of the compression flange

C_b = moment modifier from Chapter 6

C_t = accounts for top flange loading (use C_t = 1.0 for centroidal loading)

= $1 + (1.2/n)$

C_d = double curvature factor (compression in both flanges)

= $1 + (M_S/M_L)^2$

M_S = smallest moment causing compression in each flange

M_L = largest moment causing compression in each flange

The C_d factor varies between 1.0 and 2.0 and is applied only to the brace closest to the inflection point. The term $(2N_iC_t)$ can be conservatively approximated as 10 for any number of nodal forces and 4 for relative bracing and (C_bP_f) can be approximated by M_u / h which simplifies Equation C3.3-3 to the stiffness requirements given by Equations 3.3-8 and 3.3-10. Equation C3.3-3 can be used in lieu of Equations 3.3-8 and 3.3-10.

The brace strength requirement for relative bracing is

$$P_{br} = 0.004 M_u C_t C_d / h_o \quad \text{(C3.3-4a)}$$

And for nodal bracing

$$P_{br} = 0.01 M_u C_t C_d / h_o \quad \text{(C3.3-4b)}$$

They are based on an assumed initial lateral displacement of the compression flange of $0.002L_b$. The brace strength requirements of Equations 3.3-7 and 3.3-9 are derived from Equations C3.3-4a and C3.3-4b assuming top flange loading $(C_t = 2)$. Equations C3.3-4a and C3.3-4b can be used in lieu of Equations 3.3-7 and 3.3-9, respectively.

C3.3.4.2 **Torsional Bracing.** Torsional bracing can either be attached continuously along the length of the beam (for example, metal deck or slabs) or be located at discrete points along the length of the member (for example, cross frames). Torsional bracing attached to the tension flange is just as effective as a brace attached at mid depth or the compression flange. Partially restrained connections can be used if their stiffness is considered in evaluating the torsional brace stiffness.

The torsional brace requirements are based on the buckling strength of a beam with a continuous torsional brace along its length developed by Taylor and Ojalvo (1966) and modified for cross-section distortion by Yura (1993).

$$M_u \le M_{cr} = \sqrt{(C_{bu} M_o)^2 + \frac{C_b^2 EI_y \overline{\beta}_T}{2C_{tt}}} \quad \text{(C3.3-5)}$$

The term $(C_{bu}M_o)$ is the buckling strength of the beam without torsional bracing. $C_{tt} = 1.2$ when there is top flange loading and $C_{tt} = 1.0$ for centroidal loading. $\overline{\beta}_T = n\beta_T / L$ is the continuous torsional brace stiffness per unit length or its

equivalent when n nodal braces, each with a stiffness β_T, are used along the span L and the 2 accounts for initial out-of-straightness. Neglecting the un-braced beam buckling term gives a conservative estimate of the torsional brace stiffness requirement (Equation 3.3-13). A more accurate estimate of the brace requirements can be obtained by replacing M_u with $(M_u - C_{bu}M_o)$ in Equations 3.3-11 and 3.3-13. The β_{sec} term in Equations 3.3-12, 3.3-14 and 3.3-15 accounts for cross-section distortion. A web stiffener at the brace point reduces cross-sectional distortion and improves the effectiveness of a torsional brace. When a cross frame is attached near both flanges or a diaphragm is approximately the same depth as the girder, then web distortion will be insignificant so β_{sec} equals infinity. The required bracing stiffness, β_{Tb}, given by Equation 3.3-12 was obtained by solving the following expression that represents the brace system stiffness including distortion effects:

$$\frac{1}{\beta_T} = \frac{1}{\beta_{Tb}} + \frac{1}{\beta_{sec}} \quad \text{(C3.3-6)}$$

The brace moment requirements are based on an assumed initial twist of $0.002L_b/h_{o.}$

Parallel chord trusses with both chords extended to the end of the span and attached to supports can be treated like beams. In Equations 3.3-7 through 3.3-11, M_u may be taken as the maximum compressive chord force times the depth of the truss to determine the brace strength and stiffness requirements. Cross-section distortion effects, β_{sec}, need not be considered when full-depth cross frames are used for bracing. When either chord does not extend to the end of the span, consideration should be given to control twist near the ends of the span by the use of cross frames or ties.

CHAPTER 4
TENSION MEMBERS

SECTION C4.1
DESIGN TENSILE STRENGTH

Due to strain hardening, a ductile steel bar loaded in axial tension can resist, without fracture, a force greater than the product of its gross area and its coupon yield stress. However, excessive elongation of a tension member due to uncontrolled yielding of its gross area not only marks the limit of its usefulness, but can precipitate failure of the structural system of which it is a part. On the other hand, depending upon the reduction of area and other mechanical properties of the steel, the member can fail by fracture of the net area at a load smaller than required to yield the gross area. Hence, general yielding of the gross area and fracture of the net area both constitute failure limit states.

The length of the member in the net area is negligible relative to the total length of the member. As a result, the strain hardening condition is quickly reached and yielding of the net area at fastener holes does not constitute a limit state of practical significance.

SECTION C4.2
BUILT-UP MEMBERS

The slenderness ratio L/r of tension members other than rods, HSS, or straps should preferably not exceed the limiting value of 300. This slenderness limit recommended for tension members is not essential to the structural integrity of such members; it merely assures a degree of stiffness such that undesirable lateral movement (“slapping” or vibration) will be unlikely.

SECTION C4.3
PIN-CONNECTED MEMBERS AND EYEBARS

Forged eyebars have generally been replaced by pin-connected plates or eyebars thermally cut from plates. Provisions for the proportioning of eyebars contained in the LRFD Code requirements are based upon standards evolved from long experience with forged eyebars. Through extensive destructive testing, eyebars have been found to provide balanced designs when they are thermally cut instead of forged. The somewhat more conservative rules for pin-connected members of non-uniform cross section and those not having enlarged “circular” heads are likewise based on the results of experimental research.

Somewhat stockier proportions are provided for eyebars and pin-connected members fabricated from steel having a yield stress greater than 485 MPa, in order to eliminate any possibility of their “dishing” under the higher design stress.

CHAPTER 5
COLUMN AND OTHER COMPRESSION MEMBERS

SECTION C5.1
EFFECTIVE LENGTH AND SLENDERNESS LIMITATIONS

C5.1.1 **Effective Length.** The Commentary on Section 3.2 regarding frame stability and effective length factors applies here. Further analytic methods, formulas, charts, and references for the determination of effective length are provided in Chapter 15 of the SSRC Guide (Galambos, 1998).

SECTION C5.2
DESIGN COMPRESSIVE STRENGTH FOR FLEXURAL BUCKLING

Equations 5.2-2 and 5.2-3 are based on a reasonable conversion of research data into design equations and are essentially the same curve as column-strength curve 2P of the Structural Stability Research Council which is based on an initial out-of-straightness curve of $l/1500$ (Bjorhovde, 1972 and 1988; Galambos, 1998; Tide, 1985).

Equations 5.2-2 and 5.2-3 can be restated in terms of the more familiar slenderness ratio Kl/r. First, Equation 5.2-2 is expressed in exponential form as:

$$F_{cr} = [\exp(-0.419\lambda_c^2)]F_y \qquad \text{(C5.2-1)}$$

Note that exp(x) is identical to e^x. Substitution of λ_c according to definition of λ_c in Section 5.2 gives,

For $\dfrac{Kl}{r} \leq 4.71\sqrt{\dfrac{E}{F_y}}$

$$F_{cr} = \left\{\exp\left[-0.0424\frac{F_y}{E}\left(\frac{Kl}{r}\right)^2\right]\right\}F_y \qquad \text{(C5.2-2)}$$

For $\dfrac{Kl}{r} > 4.71\sqrt{\dfrac{E}{F_y}}$

$$F_{cr} = \frac{0.877\pi^2 E}{\left(\dfrac{Kl}{r}\right)^2} \qquad \text{(C5.2-3)}$$

SECTION C5.3
DESIGN COMPRESSIVE STRENGTH FOR FLEXURAL-TORSIONAL BUCKLING

Torsional buckling of symmetric shapes and flexural-torsional buckling of unsymmetric shapes are failure modes usually not considered in the design of hot-rolled columns. They generally do not govern, or the critical load differs very little from the weak axis planar buckling load. Such buckling loads may,

however, control the capacity of symmetric columns made from relatively thin plate elements and unsymmetric columns. The AISC Design Guide, *Torsional Analysis of Structural Steel Members* (Seaburg and Carter, 1997) provides an overview of the fundamentals and basic theory of torsional loading for structural steel members. Design examples are also included.

Tees that conform to the limits in Table C5.3-1 need not be checked for flexural-torsional buckling.

A simpler and more accurate design strength for the special case of tees and double-angles is based on Galambos (1991) wherein the y-axis of symmetry flexural-buckling strength component is determined directly from the column formulas.

The separate AISC *Specification for Load and Resistance Factor Design of Single-Angle Members* contains detailed provisions not only for the limit state of compression, but also for tension, shear, flexure, and combined forces.

The equations in 5.3-2 for determining the flexural-torsional elastic buckling loads of columns are derived in texts on structural stability. Since these equations for flexural-torsional buckling apply only to elastic buckling, they must be modified for inelastic buckling when $F_{cr} > 0.5F_y$. This is accomplished through the use of the equivalent slenderness factor $\lambda_e = \sqrt{F_y / F_e}$.

TABLE C5.3-1
Limiting Proportions for Tees

Shape	Ratio of Full Flange Width to Profile Depth	Ratio of Flange Thickness to Web or Stem Thickness
Built-up tees	≥ 0.50	≥ 1.25
Rolled tees	≥ 0.50	≥ 1.10

SECTION C5.4
BUILT-UP MEMBERS

Requirements for detailing and design of built-up members, which cannot be stated in terms of calculated stress, are based upon judgment and experience.

The longitudinal spacing of connectors connecting components of built-up compression members must be such that the slenderness ratio l / r of individual shapes does not exceed three-fourths of the slenderness ratio of the entire member. Additional requirements are imposed for built-up members consisting of angles. However, these minimum requirements do not necessarily ensure that the effective slenderness ratio of the built-up member is equal to that for the built-up member acting as a single unit. Section 5.4 gives formulas for modified slenderness ratios that are based on research and take into account the effect of shear deformation in the connectors. Equation 5.4-1 for snug tight intermediate connectors is empirically based on test results (Zandonini, 1985). Equation 5.4-2 is derived from theory and verified by test data. In both cases the end connection must be welded or slip-critical bolted (Aslani and Goel, 1991). The connectors must be designed to resist the shear forces which develop in the buckled member. The shear stresses are highest where the slope of the buckled shape is

maximum (Bleich, 1952).

Maximum fastener spacing less than that required for strength may be needed to ensure a close fit over the entire faying surface of components in continuous contact. Specific requirements are given for weathering steel members exposed to atmospheric corrosion (Brockenbrough, 1983).

The provisions governing the proportioning of perforated cover plates are based upon extensive experimental research (Stang and Jaffe, 1948).

CHAPTER 6
BEAMS AND OTHER FLEXURAL MEMBERS

SECTION C6.1
DESIGN FOR FLEXURE

C6.1.1 **Yielding**. The bending strength of a laterally braced compact section is the plastic moment M_p. If the shape has a large shape factor (ratio of plastic moment to the moment corresponding to the onset of yielding at the extreme fiber), significant inelastic deformation may occur at service load if the section is permitted to reach M_p at factored load. The limit of $1.5M_y$ at factored load will control the amount of inelastic deformation for sections with shape factors greater than 1.5. This provision is not intended to limit the plastic moment of a hybrid section with a web yield stress lower than the flange yield stress. Yielding in the web does not result in significant inelastic deformations. In hybrid sections, $M_y = F_{yf} S$.

Lateral-torsional buckling cannot occur if the moment of inertia about the bending axis is equal to or less than the moment of inertia out of plane. Thus, for shapes bent about the minor axis and shapes with $I_x = I_y$, such as square or circular shapes, the limit state of lateral-torsional buckling is not applicable and yielding controls if the section is compact.

Three limit states must be investigated to determine the moment capacity of flexural members: lateral-torsional buckling (LTB), local buckling of the compression flange (FLB), and local buckling of the web (WLB). These limit states depend, respectively, on the beam slenderness ratio L_b/r_y, the width-thickness ratio b/t of the compression flange and the width-thickness ratio h/t_w of the web. For convenience, all three measures of slenderness are denoted by (λ).Variations in M_n with L_b are shown in Figure C6.1-1. Values of λ_p for FLB and WLB produce a compact section with a rotation capacity of about three (after reaching M_p) before the onset of local buckling, and therefore meet the requirements for plastic analysis of load effects (Commentary Section 2.5). On the other hand, values of λ_p for LTB do not allow plastic analysis because they do not provide rotation capacity beyond that needed to develop M_p. Instead $L_b \leq L_{pd}$ (Section 6.1.3) must be satisfied. Analyses to include restraint effects of adjoining elements are discussed in Galambos (1998). Analysis of the lateral stability of members with shapes not covered in this Chapter must be performed according to the available literature (Galambos, 1998).

See the Commentary for Section 2.5 for the discussion of the equation regarding the bending capacity of circular sections.

C6.1.2.1 **Doubly Symmetric Shapes and Channels with $L_b \leq L_r$**. The basic relationship between nominal moment M_n and unbraced length L_b is shown in Figure C6.1-1 for a compact section with $C_b = 1.0$. There are four principal zones defined on the basic curve by L_{pd}, L_p, *and* L_r. Equation 6.1-4 defines the maximum unbraced length L_p to reach M_p with uniform moment. Elastic lateral-torsional buckling will occur when the unbraced length is greater than L_r given by Equation 6.1-6. Equation 6.1-2 defines the inelastic lateral-torsional buckling as a straight line between the defined limits L_p *and* L_r. Buckling strength in the elastic region $L_b > L_r$ is given by Equation 6.1-13 for I-shaped members.

For other moment diagrams, the lateral buckling strength is obtained by multiplying the basic strength by C_b as shown in Figure C6.1-1. The maximum M_n, however, is limited to M_p. Note that L_p given by Equation 6.1-4 is merely a definition which has physical meaning when $C_b = 1.0$. For C_b greater than 1.0, larger unbraced lengths are permitted to reach M_p as shown by the curve for $C_b > 1.0$. For design, this length could be calculated by setting Equation 6.1-2 equal to M_p and solving this equation for L_b using the desired C_b value.

The equation

$$C_b = 1.75+1.05(M_1/M_2) +0.3(M_1/M_2)^2 \leq 2.3 \qquad \text{(C6.1-1)}$$

has been used since 1961 to adjust the flexural-torsional buckling equation for variations in the moment diagram within the unbraced length. This equation is applicable only to moment diagrams that are straight lines between braced points. Another equation

$$C_b = \frac{1}{0.6-0.4\dfrac{M_1}{M_2}} \leq 2.5 \qquad \text{(C6.1-2)}$$

fits the average value theoretical solutions when the beams are bent in reverse curvature and also provides a reasonable fit to the theory. If the maximum moment within the unbraced segment is equal to or larger than the end moment, $C_b = 1.0$ is used.

The equations above can be easily misinterpreted and misapplied to moment diagrams that are not straight within the unbraced segment. Kirby and Nethercot (1979) presented an equation which applies to various shapes of moment diagrams within the unbraced segment. Their equation has been adjusted slightly to the following

$$C_b = \frac{12.5M_{\max}}{2.5M_{\max} + 3M_A + 4M_B + 3M_C} \qquad \text{(C6.1-3)}$$

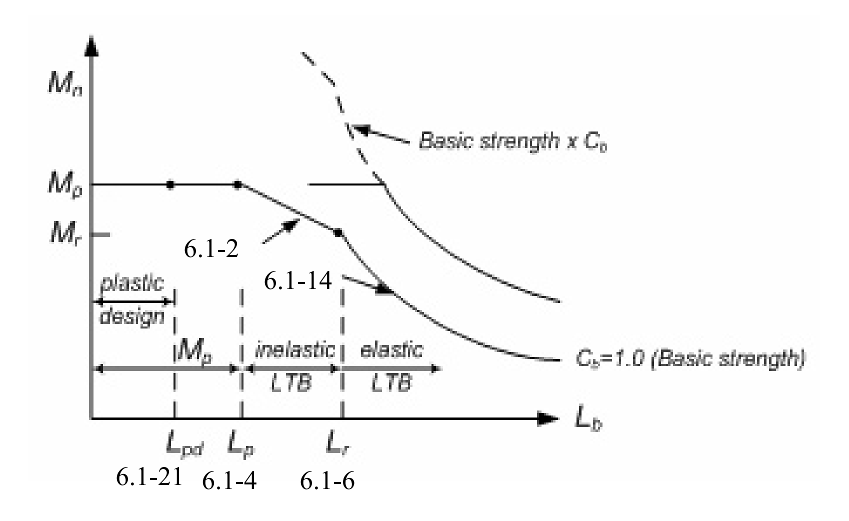


Figure. C6.1-1. Nominal moment as a function of unbraced length and moment gradient.

This equation gives more accurate solutions for fixed-end beams, and the adjusted equation reduces exactly to Equation C6.1-2 for a straight line moment diagram in single curvature. The C_b equation used in the SBC 306 is shown in Figure C6.1-2 for straight line moment diagrams. Other moment diagrams along with exact theoretical solutions in the SSRC Guide (Galambos, 1998) show good comparison with the new equation. The absolute values of the three interior quarter-point moments plus the maximum moment, regardless of its location are used in the equation. The maximum moment in the unbraced segment is always used for comparison with the resistance. The length between braces, not the distance to inflection points and C_b are used in the resistance equation.

It is still satisfactory to use the former C_b factor, Equation C6.1-1, for straight line moment diagrams within the unbraced length.

The elastic strength of hybrid beams is identical to homogeneous beams. The strength advantage of hybrid sections becomes evident only in the inelastic and plastic slenderness ranges.

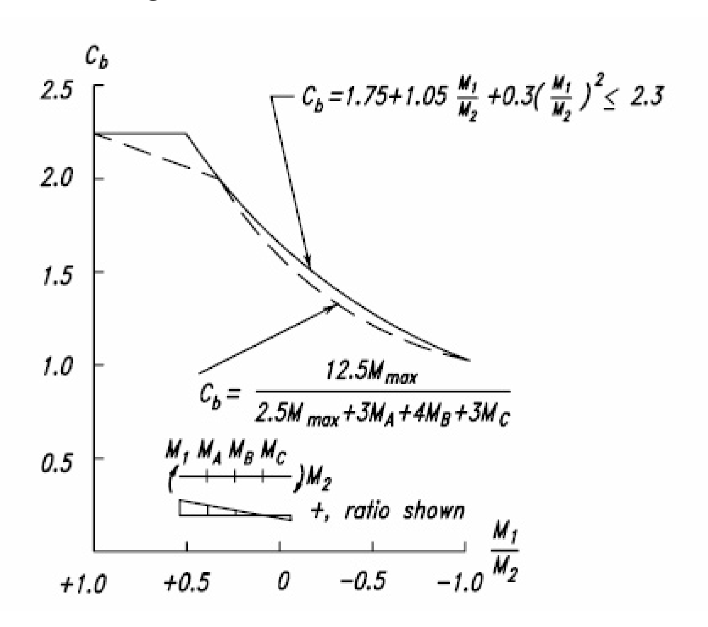


Figure. C6.1-2. **C_b for a straight line moment diagram-prismatic beam.**

C6.1.2.2 **Doubly Symmetric Shapes and Channels with $L_b > L_r$.** The equation given in the SBC 306 assumes that the loading is applied along the beam centroidal axis. If the load is placed on the top flange and the flange is not braced, there is a tipping effect that reduces the critical moment; conversely, if the load is suspended from the bottom flange and is not braced, there is a stabilizing effect which increases the critical moment (Galambos, 1998). For unbraced top flange loading, the reduced critical moment may be conservatively approximated by setting the warping buckling factor X_2 to zero.

An effective length factor of unity is implied in these critical moment equations to represent a worst case pinned-pinned unbraced segment. Including consideration of any end restraint of the adjacent segments on the critical segment can increase its buckling capacity. The effects of beam continuity on lateral-torsional buckling

have been studied and a simple and conservative design method, based on the analogy of end-restrained nonsway columns with an effective length factor less than one, has been proposed (Galambos, 1998).

C6.1.2.3 **Tees and Double-Angles**. The lateral-torsional buckling strength (LTB) of singly symmetric tee beams is given by a fairly complex formula (Galambos, 1998). Equation 6.1-15 is a simplified formulation based on Kitipornchai and Trahair (1980). See also Ellifritt, Wine, Sputo, and Samuel (1992).

The C_b used for I-shaped beams is unconservative for tee beams with the stem in compression. For such cases $C_b = 1.0$ is appropriate. When beams are bent in reverse curvature, the portion with the stem in compression may control the LTB resistance even though the moments may be small relative to other portions of the unbraced length with $C_b \approx 1.0$. This is because the LTB strength of a tee with the stem in compression may be only about one-fourth of the capacity for the stem in tension. Since the buckling strength is sensitive to the moment diagram, C_b has been conservatively taken as 1.0. In cases where the stem is in tension, connection details should be designed to minimize any end restraining moments which might cause the stem to be in compression.

C6.1.2.4 **Design by Plastic Analysis**. Equation 6.1-17 sets a limit on unbraced length adjacent to a plastic hinge for plastic analysis. There is a substantial increase in unbraced length for positive moment ratios (reverse curvature) because the yielding is confined to zones close to the brace points (Yura et al., 1978).

Equation 6.1-18 is an equation in similar form for solid rectangular bars and symmetric box beams. Equations 6.1-17 and 6.1-18 assume that the moment diagram within the unbraced length next to plastic hinge locations is reasonably linear. For nonlinear diagrams between braces, judgment should be used in choosing a representative ratio.

Equations 6.1-17 and 6.1-18 were developed to provide rotation capacities of at least 3.0, which are sufficient for most applications (Yura et al., 1978). When inelastic rotations of 7 to 9 are deemed appropriate in areas of high seismicity, as discussed in Commentary Section 2.5, Equation 6.1-17 would become:

$$L_{pd} = 0.086\left(\frac{E}{F_y}\right)r_y \qquad \text{(C6.1-4)}$$

SECTION C6.2
DESIGN FOR SHEAR

For unstiffened webs $k_v = 5.0$, therefore

$$1.10\sqrt{Ek_v / F_{yw}} = 2.45\sqrt{E / F_{yw}}, \text{and}\, 1.37\sqrt{Ek_v / F_{yw}} = 3.07\sqrt{E / F_{yw}}$$

For webs with $h / t_w \leq 1.10\sqrt{Ek_v / F_{yw}}$, the nominal shear strength V_n is based on shear yielding of the web, Equation 6.2-1. This h/t_w limit was determined by setting the critical stress causing shear buckling F_{cr} equal to the yield stress of the web F_{yw} in Equation 35 of Cooper, Galambos, and Ravindra (1978) and

Timoshenko and Gere (1961). When, $h/t_w > 1.10\sqrt{Ek_v/F_{yw}}$, the web shear strength is based on buckling. Basler (1961) suggested taking the proportional limit as 80 percent of the yield stress of the web. This corresponds to $h/t_w = (1.10/0.8)\sqrt{Ek_v/F_{yw}}$. Thus, when $h/t_w > 1.10\sqrt{Ek_v/F_{yw}}$, the web strength is determined from the elastic buckling stress given by Equation 6 of Cooper et al. (1978) and Timoshenko and Gere (1961):

$$F_{cr} = \frac{\pi^2 E k_v}{12(1-v^2)(h/t_w)^2} \qquad \text{(C6.2-1)}$$

The nominal shear strength, given by Equation 6.2-3, was obtained by multiplying F_{cr} by the web area and using $E = 200000\ MPa$ and $v = 0.3$. A straight line transition, Equation 6.2-2 is used between the limits

$$1.10\sqrt{Ek_v/F_{yw}} \text{ and } 1.37\sqrt{Ek_v/F_{yw}}$$

When designing plate girders, thicker unstiffened webs will frequently be less costly than lighter stiffened web designs because of the additional fabrication. If a stiffened girder design has economic advantages, the tension field method in Chapter 7 will require fewer stiffeners.

The equations in this section were established assuming monotonically increasing loads. If a flexural member is subjected to load reversals causing cyclic yielding over large portions of a web, such as may occur during a major earthquake, special design considerations may apply (Popov, 1980).

SECTION C6.3
WEB-TAPERED MEMBERS

C6.3.1 **General Requirements**. The design of wide-flange columns with a single web taper and constant flanges follows the same procedure as for uniform columns according to Section 5.2, except the column slenderness parameter λ_c for major axis buckling is determined for a slenderness ratio $K_\gamma L/r_{ox}$, and for minor axis buckling for K_L/r_{oy}, where K_γ is an effective length factor for tapered members, K is the effective length factor for prismatic members, and r_{ox} *and* r_{oy} are the radii of gyration about the x and the y axes, respectively, taken at the smaller end of the tapered member.

For stepped columns or columns with other than a single web taper, the elastic critical stress is determined by analysis or from data in reference texts or research reports (Chapters 11 and 13 in Timoshenko and Gere (1961), Bleich (1952), and Kitipornchai and Trahair (1980)), and then the same procedure of using λ_{eff} is utilized in calculating the factored resistance.

This same approach is recommended for open section built-up columns (columns with perforated cover plates, lacing, and battens) where the elastic critical buckling stress determination must include a reduction for the effect of shear. Methods for calculating the elastic buckling strength of such columns are given in Chapter 12 of the SSRC Guide (Galambos, 1998) and in Timoshenko and Gere (1961) and Bleich (1952).

C6.3.3 **Design Compressive Strength**. The approach in formulating $F_{a\gamma}$ of tapered columns is based on the concept that the critical stress for an axially loaded tapered column is equal to that of a prismatic column of different length, but of the same cross section as the smaller end of the tapered column. This has resulted in an equivalent effective length factor K_γ for a tapered member subjected to axial compression (Lee, Morrell, and Ketter, 1972). This factor, which is used to determine the value of S in Equations 6.3-2 and λ_c in Equation 5.2-3, can be determined accurately for a symmetrical rectangular rigid frame comprised of prismatic beams and tapered columns.

With modifying assumptions, such a frame can be used as a mathematical model to determine with sufficient accuracy the influence of the stiffness $\Sigma(I/b)_g$ of beams and rafters which afford restraint at the ends of a tapered column in other cases such as those shown in Figure C6.3-1 from Equations 6.3-2 and 5.2-3, the critical load P_{cr} can be expressed as $\pi^2 EI_\circ/(K_\gamma l)^2$. The value of K_γ can be obtained by interpolation, using the appropriate chart from Lee et al. (1972) and restraint modifiers G_T *and* G_B. In each of these modifiers the tapered column, treated as a prismatic member having a moment of inertia I_o, computed at the smaller end, and its actual length l, is assigned the stiffness I_o / l, which is then divided by the stiffness of the restraining members at the end of the tapered column under consideration.

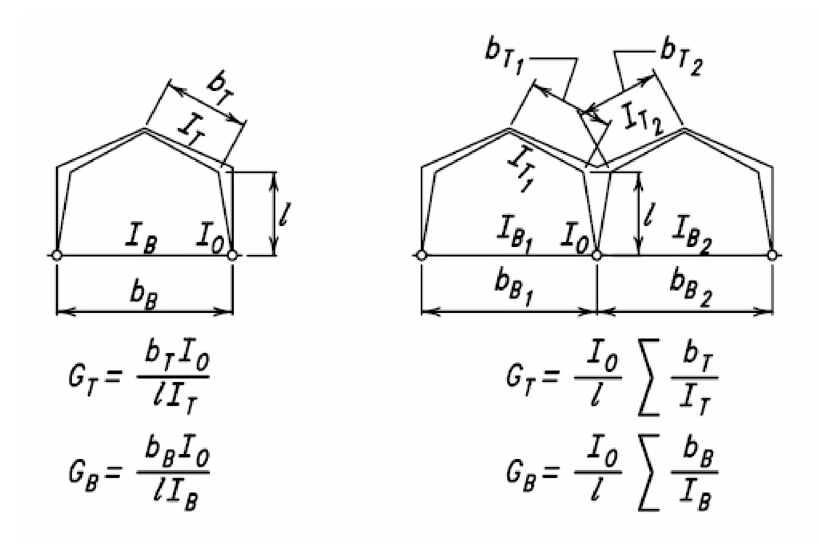


Figure. C6.3-1. Restraint modifiers for tapered columns.

C6.3.4 **Design Flexural Strength**. The development of the design bending stress for tapered beams follows closely with that for prismatic beams. The basic concept is to replace a tapered beam by an equivalent prismatic beam with a different length, but with a cross section identical to that of the smaller end of the tapered beam (Lee et al., 1972). This has led to the modified length factors h_s and h_w in Equations 6.3-6 and 6.3-7.

Equations 6.3-6 and 6-3-7 are based on total resistance to lateral buckling, using both St. Venant and warping resistance. The factor B modifies the basic $F_{b\gamma}$ to members, which are continuous past lateral supports. Categories a, b, and c of Section 6.3.4 usually apply; however, it is to be noted that they apply only when

the axial force is small and adjacent unbraced segments are approximately equal in length. For a single member, or segments which do not fall into category a, b, c, or d, the recommended value of B is unity. The value of B should also be taken as unity when computing the value of $F_{b\gamma}$ to obtain M_n to be used in Equations 8.1-1 and 3.1-1, since the effect of moment gradient is provided for by the factor C_m. The background material is given in WRC Bulletin No. 192 (Morrell and Lee, 1974).

SECTION C6.4
BEAMS AND GIRDERS WITH WEB OPENINGS

Web openings in structural floor members may be necessary to accommodate various mechanical, electrical, and other systems. Strength limit states, including local buckling of the compression flange, web, and tee-shaped compression zone above or below the opening, lateral buckling and moment-shear interaction, or serviceability may control the design of a flexural member with web openings. The location, size, and number of openings are important and empirical limits for them have been identified. One general procedure for assessing these effects and the design of any needed reinforcement for both steel and composite beams is given in Darwin (1990) and in ASCE Task Committee on Design Criteria for Composite Structures in Steel and Concrete (1992 and 1992a).

CHAPTER 7
PLATE GIRDERS

SECTION C7.2
DESIGN FLEXURAL STRENGTH

In previous versions of the AISC Specification a coefficient of $0.0005a_r$ was used in R_{PG} based on the work of Basler (1961). This value is valid for $a_r \leq 2$. In that same paper, Basler developed a more general coefficient, applicable to all ratios of A_w/A_f which has been adopted because application of the previous equation to sections with large a_r values gives unreasonable results. An arbitrary limit of $a_r \leq 10$ is imposed so that the R_{PG} expression is not applied to sections approaching a tee shape.

CHAPTER 8
MEMBERS UNDER COMBINED FORCES AND TORSION

SECTION C8.1
SYMMETRIC MEMBERS SUBJECT TO BENDING AND AXIAL FORCE

Equations 8.1-1a and 8.1-1b are simplifications and clarifications of similar equations used in the AISC ASD Specification since 1961. Previously, both equations had to be checked. In the new formulation the applicable equation is governed by the value of the first term, $P_u / \phi P_n$. For bending about one axis only, the equations have the form shown in Figure C8.1-1.

The first term $P_u/\phi P_n$ has the same significance as the axial load term f_a / F_a in Equations 8.1-1 of the SBC 306. This means that for members in compression P_n must be based on the largest effective slenderness ratio Kl / r. In the development of Equations 8.1-1a and 8.1-1b, a number of alternative formulations were compared to the exact inelastic solutions of 82 side-sway cases reported in Kanchanalai (1977). In particular, the possibility of using Kl/r as the actual column length *(K = 1)* in determining P_n, combined with an elastic second order moment Mu, was studied. In those cases where the true P_n based on Kl/r, with $K = 1.0$, was in the inelastic range, the errors proved to be unacceptably large without the additional check that $P_u \leq \phi_c P_n$, P_n being based on effective length. Although deviations from exact solutions were reduced, they still remained high.

In summary, it is not possible to formulate a safe general interaction equation for compression without considering effective length directly (or indirectly by a

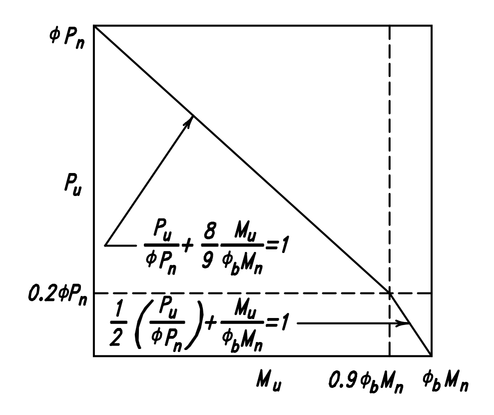


Figure. C8.1-1. Beam-column interaction equations

second equation). Therefore, the requirement that the nominal compressive strength P_n be based on the effective length *KL* in the general equation is continued in the LRFD Specification as it has been in the AISC ASD Specification since 1961. It is not intended that these provisions be applicable to limit nonlinear secondary flexure that might be encountered in large amplitude earthquake stability design (ATC, 1978).

The defined term M_u is the maximum moment in a member. In the calculation of this moment, inclusion of beneficial second order effects of tension is optional.

But consideration of detrimental second order effects of axial compression and translation of gravity loads is required. Provisions for calculation of these effects are given in Chapter 3.

The interaction equations in Section 8.3 have been recommended for bi-axially loaded H and wide flange shapes in Galambos (1998) and Springfield (1975). These equations which can be used only in braced frames represent a considerable liberalization over the provisions given in Section 8.1; it is, therefore, also necessary to check yielding under service loads, using the appropriate load and resistance factors for the serviceability limit state in Equation 8.1-1a or 8.1-1b with $M_{ux} = S_xF_y$ and $M_{uy} = S_yF_y$. Section 8.3 also provides interaction equations for rectangular box-shaped beam-columns. These equations are taken from Zhou and Chen (1985).

SECTION C8.2
UNSYMMETRIC MEMBERS AND MEMBERS UNDER TORSION AND COMBINED TORSION, FLEXURE, SHEAR, AND/OR AXIAL FORCE

This section deals with types of cross sections and loadings not covered in Section 8.1, especially where torsion is a consideration. For such cases it is recommended to perform an elastic analysis based on the theoretical numerical methods available from the literature for the determination of the maximum normal and shear stresses, or for the elastic buckling stresses. In the buckling calculations an equivalent slenderness parameter is determined for use in Equation 5.2-2 or 5.2-3, as follows:

$$\lambda_e = \sqrt{F_y / F_e}$$

where F_e is the elastic buckling stress determined from a stability analysis. This procedure is similar to that of Section 5.3.

For the analysis of members with open sections under torsion refer to AISC (1997).

CHAPTER 9
COMPOSITE MEMBERS

SECTION C9.1
DESIGN ASSUMPTIONS AND DEFINITIONS

Force Determination. Loads applied to an unshored beam before the concrete has hardened are resisted by the steel section alone, and only loads applied after the concrete has hardened are considered as resisted by the composite section. It is usually assumed for design purposes that concrete has hardened when it attains 75 percent of its design strength. In beams properly shored during construction, all loads may be assumed as resisted by the composite cross section. Loads applied to a continuous composite beam with shear connectors throughout its length, after the slab is cracked in the negative moment region, are resisted in that region by the steel section and by properly anchored longitudinal slab reinforcement.

For purposes of plastic analysis all loads are considered resisted by the composite cross section, since a fully plastic strength is reached only after considerable yielding at the locations of plastic hinges.

Elastic Analysis. The use of constant stiffness in elastic analyses of continuous beams is analogous to the practice in reinforced concrete design. The stiffness calculated using a weighted average of moments of inertia in the positive moment region and negative moment regions may take the following form:

$$I_t = aI_{pos} + bI_{neg}$$

where

I_{pos} = effective moment of inertia for positive moment, mm^4

I_{neg} = negative moment of inertia for negative moment, mm^4

The effective moment of inertia shall be based on the cracked transformed section considering degree of composite actions. For continuous beams subjected to gravity loads only, the value of a may be taken as 0.6 and the value of b may be taken as 0.4. For the case of composite beams in moment resisting frames, the value of *a* and *b* may be taken as 0.5.

Plastic Analysis. For composite beams with shear connectors, plastic analysis may be used only when the steel section in the positive moment region has a compact web, i.e., $h/t_w \leq 3.76\sqrt{E/F_{yf}}$, and when the steel section in the negative moment region is compact, as required for steel beams alone. No compactness limitations are placed on encased beams, but plastic analysis is permitted only if the direct contribution of concrete to the strength of sections is neglected; the concrete is relied upon only to prevent buckling.

Plastic Stress Distribution for Positive Moment. Plastic stress distributions are described in Commentary Section C9.3, and a discussion of the composite participation of slab reinforcement is presented.

Plastic Stress Distribution for Negative Moment. Plastic stress distributions are described in Commentary Section C9.3.

Elastic Stress Distribution. The strain distribution at any cross section of a composite beam is related to slip between the structural steel and concrete elements. Prior to slip, strain in both steel and concrete is proportional to the distance from the neutral axis for the elastic transformed section. After slip, the strain distribution is discontinuous, with a jump at the top of the steel shape. The strains in steel and concrete are proportional to distances from separate neutral axes, one for steel and the other for concrete.

Partially Composite Beam.

Fully Composite Beam. Either the tensile yield strength of the steel section or the compressive stress of the concrete slab governs the maximum flexural strength of a fully composite beam subjected to a positive moment. The tensile yield strength of the longitudinal reinforcing bars in the slab governs the maximum flexural strength of a fully composite beam subjected to a negative moment. When shear connectors are provided in sufficient numbers to fully develop this maximum flexural strength, any slip that occurs prior to yielding is minor and has negligible influence both on stresses and stiffness.

Concrete-Encased Beam. When the dimensions of a concrete slab supported on steel beams are such that the slab can effectively serve as the flange of a composite T-beam, and the concrete and steel are adequately tied together so as to act as a unit, the beam can be proportioned on the assumption of composite action.

Two cases are recognized: fully encased steel beams, which depend upon natural bond for interaction with the concrete, and those with mechanical anchorage to the slab (shear connectors), which do not have to be encased.

SECTION C9.2
COMPRESSION MEMBERS

C9.2.1 Limitations.

(1) The lower limit of four percent on the cross-sectional area of structural steel differentiates between composite and reinforced concrete columns. If the area is less than four percent, a column with a structural steel core should be designed as a reinforced concrete column.

(2) The specified minimum quantity of transverse and longitudinal reinforcement in the encasement should be adequate to prevent severe spalling of the surface concrete during fires.

(3) Very little of the supporting test data involved concrete strengths in excess of 41 MPa, even though the cylinder strength for one group of four columns was 66 MPa. Normal weight concrete is believed to have been

used in all tests. Thus, the upper limit of concrete strength is specified as 55 MPa for normal weight concrete. A lower limit of 21 MPa is specified for normal weight concrete and 28 MPa for lightweight concrete to encourage the use of good quality, yet readily available, grades of structural concrete.

(4) In addition to the work of Bridge and Roderick (1978), SSRC Task Group 20 (1979), and Galambos and Chapuis (1980), recent work by Kenny, Bruce, and Bjorhovde (1994) has shown that due to concrete confinement effects, the previous limitation of 380 MPa for the maximum steel yield stress is highly restrictive. Further, the most commonly used reinforcing steel grade has a yield stress of 415 MPa. The increase is therefore a rational recognition of material properties and structural behavior.

The 415 MPa limitations for the yield stress is very conservative for tubular composite columns, where the concrete confinement provided by the tube walls is very significant. Kenny et al. have proposed raising the value of F_y for such columns to whatever the yield stress is for the steel grade used, but not higher than 550 MPa.

(5) The specified minimum wall thicknesses are identical to those in the SBC-304. The purpose of this provision is to prevent buckling of the steel pipe or HSS before yielding.

SECTION C9.3
FLEXURAL MEMBERS

C9.3.2 **Design Strength of Beams with Shear Connectors.** This section applies to simple and continuous composite beams with shear connectors, constructed with or without temporary shores.

Positive Flexural Design Strength. Flexural strength of a composite beam in the positive moment region may be limited by the plastic strength of the steel section, the concrete slab, or shear connectors. In addition, web buckling may limit flexural strength if the web is slender and a significantly large portion of the web is in compression.

According to Table 2.5-1, local web buckling does not reduce the plastic strength of a bare steel beam if the beam depth-to-web thickness ratio is not larger than $3.76\sqrt{E/F_y}$. In the absence of web buckling research on composite beams, the same ratio is conservatively applied to composite beams. Furthermore, for more slender webs, the SBC 306 conservatively adopts first yield as the flexural strength limit. In this case, stresses on the steel section from permanent loads applied to unshored beams before the concrete has hardened must be superimposed on stresses on the composite section from loads applied to the beams after hardening of concrete. In this superposition, all permanent loads should be multiplied by the dead load factor and all live loads should be multiplied by the live load factor. For shored beams, all loads may be assumed as resisted by the composite section.

When first yield is the flexural strength limit, the elastic transformed section is used to calculate stresses on the composite section. The modular ratio $n = E/E_c$ used to determine the transformed section depends on the specified unit weight and strength of concrete.

Plastic Stress Distribution for Plastic Moment. When flexural strength is determined from the plastic stress distribution shown in Figure C9.3-1, the compression force C in the concrete slab is the smallest of:

$$C = A_{sw} F_{yw} + 2A_{sf} F_{yf} \quad \text{(C9.3-1)}$$

$$C = 0.85 f_c' A_c \quad \text{(C9.3-2)}$$

$$C = \Sigma Q_n \quad \text{(C9.3-3)}$$

For a non-hybrid steel section, Equation C9.3-1 becomes $C = A_s F_y$

where

f_c' = specified compressive strength of concrete, MPa

A_c = area of concrete slab within effective width, mm^2

A_s = area of steel cross section, mm^2

A_{sw} = area of steel web, mm^2

A_{sf} = area of steel flange, mm^2

F_y = minimum specified yield stress of steel, MPa

F_{yw} = minimum specified yield stress of web steel, MPa

F_{yf} = minimum specified yield stress of flange steel, MPa

ΣQ_n = sum of nominal strengths of shear connectors between the point of maximum positive moment and point of zero moment to either side, N

Longitudinal slab reinforcement makes a negligible contribution to the compression force, except when Equation C9.3-2 governs. In this case, the area of longitudinal reinforcement within the effective width of the concrete slab times the yield stress of the reinforcement may be added in determining C.

The depth of the compression block is

$$a = \frac{C}{0.85 f_c' b} \quad \text{(C9.3-4)}$$

where

b = effective width of concrete slab, mm

A fully composite beam corresponds to the case of C governed by the yield strength of the steel beam or the compressive strength of the concrete slab, as in Equation C9.3-1 or C9.3-2. The number and strength of shear connectors govern C for a partially composite beam as in Equation C9.3-3.

The plastic stress distribution may have the plastic neutral axis (PNA) in the web, in the top flange of the steel section or in the slab, depending on the value of C.

The nominal plastic moment resistance of a composite section in positive bending is given by the following equation and Figure C9.3-1:

$$M_n = C(d_1 + d_2) + P_y(d_3 - d_2) \quad \text{(C9.3-5)}$$

where

P_y = tensile strength of the steel section; for a non-hybrid steel section, $P_y = A_sF_y$, N.

d_1 = distance from the centroid of the compression force C in concrete to the top of the steel section, mm.

d_2 = distance from the centroid of the compression force in the steel section to the top of the steel section, mm. For the case of no compression in the steel section $d_2 = 0$.

d_3 = distance from P_y to the top of the steel section, mm.

Equation C9.3-5 is generally applicable including both non-hybrid and hybrid steel sections symmetrical about one or two axes.

Negative Flexural Design Strength. The flexural strength in the negative moment region is the strength of the steel beam alone or the plastic strength of the composite section made up of the longitudinal slab reinforcement and the steel section.

Plastic Stress Distribution for Negative Moment. When an adequately braced compact steel section and adequately developed longitudinal reinforcing bars act compositely in the negative moment region, the nominal flexural strength is determined from the plastic stress distributions as shown in Figure C9.3-2. The tensile force T in the reinforcing bars is the smaller of:

$$T = A_r F_{yr} \qquad \text{(C9.3-6)}$$

$$T = \Sigma Q_n \qquad \text{(C9.3-7)}$$

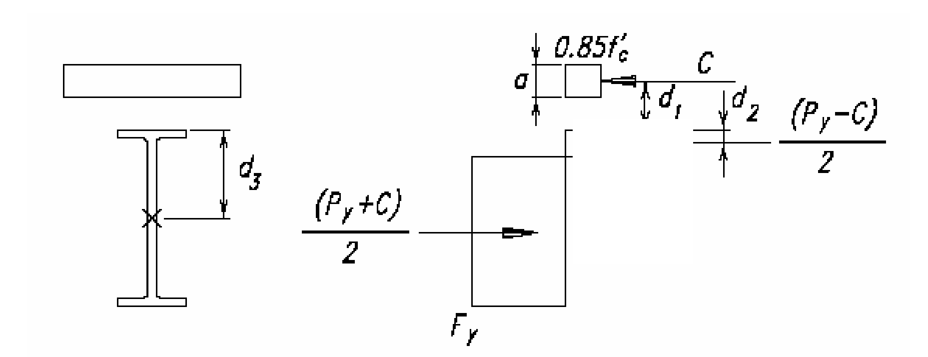


Figure. C9.3-1. Plastic stress distribution for positive moment in composite beams.

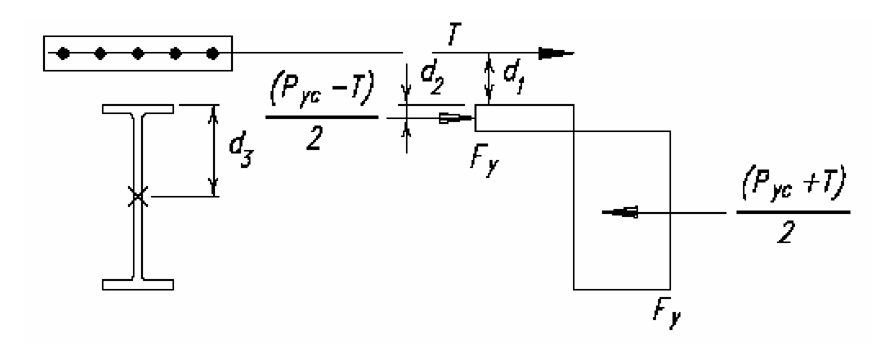


Figure. C9.3-2. Plastic stress distribution for negative moment.

where

A_r = area of properly developed slab reinforcement parallel to the steel beam and within the effective width of the slab, mm^2

F_{yr} = specified yield stress of the slab reinforcement, MPa

ΣQ_n = sum of the nominal strengths of shear connectors between the point of maximum negative moment and point of zero moment to either side, N

A third theoretical limit on T is the product of the area and yield stress of the steel section. However, this limit is redundant in view of practical limitations on slab reinforcement.

The nominal plastic moment resistance of a composite section in negative bending is given by the following equation:

$$M_n = T(d_1 + d_2) + P_{yc}(d_3 - d_2) \qquad \text{(C9.3-8)}$$

where

P_{yc} = the compressive strength of the steel section; for a non-hybrid section, $P_{yc} = A_s F_y$, N

d_1 = distance from the centroid of the longitudinal slab reinforcement to the top of the steel section, mm

d_2 = distance from the centroid of the tension force in the steel section to the top of the steel section, mm

d_3 = distance from P_{yc} to the top of the steel section, mm

Transverse Reinforcement for the Slab. Where experience has shown that longitudinal cracking detrimental to serviceability is likely to occur, the slab should be reinforced in the direction transverse to the supporting steel section. It is recommended that the area of such reinforcement should be at least 0.002 times the concrete area in the longitudinal direction of the beam and should be uniformly distributed.

C9.3.3 Design Strength of Concrete-Encased Beams. Tests of concrete-encased beams demonstrated that (1) the encasement drastically reduces the possibility of lateral-torsional instability and prevents local buckling of the encased steel, (2)

the restrictions imposed on the encasement practically prevent bond failure prior to first yielding of the steel section, and (3) bond failure does not necessarily limit the moment capacity of an encased steel beam (ASCE, 1979).

Accordingly, the SBC 306 permits three alternative design methods: one based on the first yield in the tension flange of the composite section; one based on the plastic moment capacity of the steel beam alone; and a third method based upon the plastic moment capacity of the composite section applicable only when shear connectors are provided along the steel section and reinforcement of the concrete encasement meets the specified detailing requirements. No limitations are placed on the slenderness of either the composite beam or the elements of the steel section, since the encasement effectively inhibits both local and lateral buckling.

In the method based on first yield, stresses on the steel section from permanent loads applied to unshored beams before the concrete has hardened must be superimposed on stresses on the composite section from loads applied to the beams after hardening of the concrete. In this superposition, all permanent loads should be multiplied by the dead load factor and all live loads should be multiplied by the live load factor. For shored beams, all loads may be assumed as resisted by the composite section. Complete interaction (no slip) between the concrete and steel is assumed.

The contribution of concrete to the strength of the composite section is ordinarily larger in positive moment regions than in negative moment regions. Accordingly, design based on the composite section is more advantageous in the regions of positive moments.

C9.3.4 **Strength During Construction.** When temporary shores are not used during construction, the steel beam alone must resist all loads applied before the concrete has hardened enough to provide composite action. Unshored beam deflection caused by wet concrete tends to increase slab thickness and dead load. For longer spans this may lead to instability analogous to roof ponding. An excessive increase of slab thickness may be avoided by beam camber.

When forms are not attached to the top flange, lateral bracing of the steel beam during construction may not be continuous and the un-braced length may control flexural strength, as defined in Section 6.1.

The SBC 306 does not include special requirements for a margin against yield during construction. According to Section 6.1, maximum factored moment during construction is $0.90F_yZ$ where F_yZ is the plastic moment $0.90 F_y Z \approx 0.90 \times 1.1 F_y S$. This is equivalent to approximately the yield moment, F_yS. Hence, required flexural strength during construction prevents moment in excess of the yield moment.

Load factors for construction loads should be determined for individual projects according to local conditions, using the factors stipulated in SBC 301 as a guide. As a minimum it is suggested that 1.2 be the factor for the loading from steel framing plus concrete plus formed steel deck, and a factor of 1.6 be used for the live load of workmen plus equipment which should not be taken as less than 950 N/m^2 (unfactored).

C9.3.5 **Formed Steel Deck.** Figure C9.3-3 is a graphic presentation of the terminology used in Section 9.3.5.

When studs are used on beams with formed steel deck, they may be welded directly through the deck or through pre-punched or cut-in-place holes in the deck. The usual procedure is to install studs by welding directly through the deck; however, when the deck thickness is greater than 16 gage (1.5 mm) for single thickness, or 18 (1.2 mm) gage for each sheet of double thickness, or when the total thickness of galvanized coating is greater than 0.38 kg/m^2, special precautions and procedures recommended by the stud manufacturer should be followed.

As shown in Figure C9.3-4, modern steel deck profiles with stiffeners (reinforcing rib) located along the centerline of the rib require that studs be placed off-center in the rib. Depending on the location of the stud relative to the direction of the shear transfer, for studs in the "weak position", the resulting reduction in edge distance between the stud and rib wall can lead to premature failure accompanied by punching of the stud through the steel deck. Therefore, in addition to applying the required cap of 0.75 on the reduction factor (Equation 9.3-1) for single studs in a rib, it is recommended to avoid situations where all studs may be located in the "weak position" by either alternating stud placement between the "weak" and "strong" positions or coordinating placement of studs to ensure they are all installed in the strong position.

Based on the Lehigh test data (Grant et al., 1977), the maximum spacing of steel deck anchorage to resist uplift was increased from 405 mm to 460 mm in order to accommodate current production profiles.

When metal deck includes units for carrying electrical wiring, crossover headers are commonly installed over the cellular deck perpendicular to the ribs. They create trenches which completely or partially replace sections of the concrete slab above the deck. These trenches, running parallel to or transverse to a composite beam, may reduce the effectiveness of the concrete flange. Without special provisions to replace the concrete displaced by the trench, the trench should be considered as a complete structural discontinuity in the concrete flange.

When trenches are parallel to the composite beam, the effective flange width should be determined from the known position of the trench.

Trenches oriented transverse to composite beams should, if possible, be located in areas of low bending moment and the full required number of studs should be placed between the trench and the point of maximum positive moment. Where the trench cannot be located in an area of low moment, the beam should be designed as non-composite.

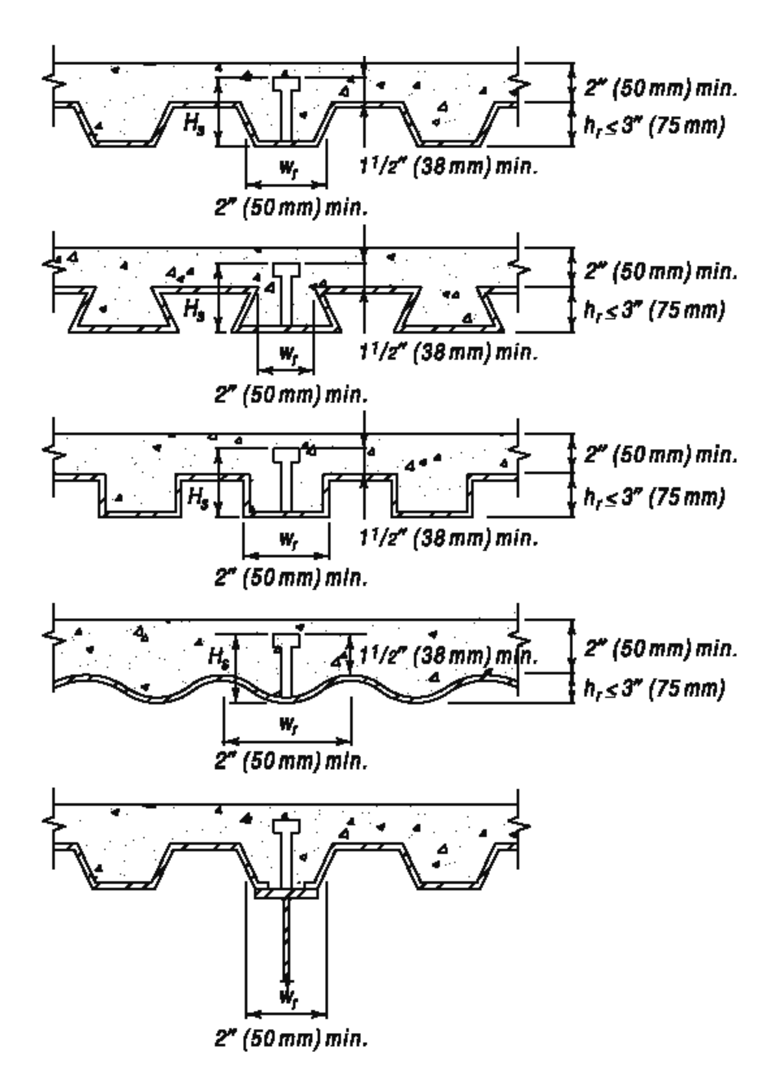


Figure. C9.3-3 Steel deck limits.

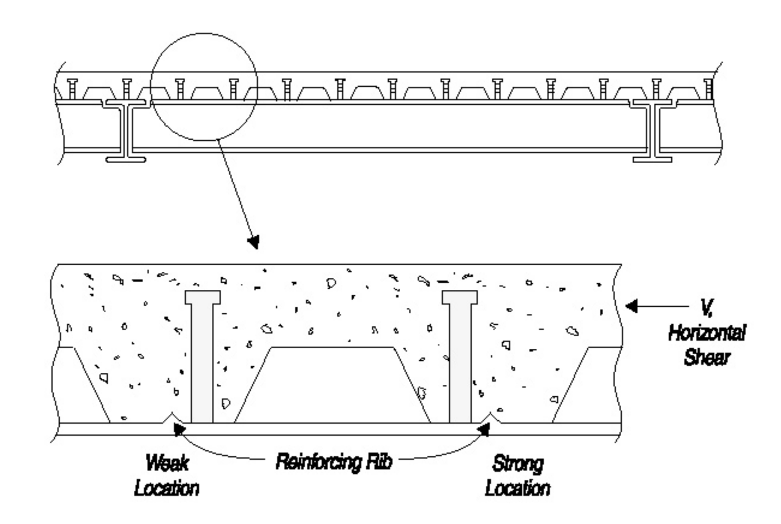


Figure. C9.3-4 Alternative shear stud positions in rib decked profiles.

SECTION C9.4
COMBINED COMPRESSION AND FLEXURE

The last paragraph in Section 9.4 provides a transition from beam-columns to beams. It involves bond between the steel section and concrete. Section 9.3 for beams requires either shear connectors or full, properly reinforced encasement of the steel section. Furthermore, even with full encasement, it is assumed that bond is capable of developing only the moment at first yielding in the steel of the composite section. No test data are available on the loss of bond in composite beam-columns. However, consideration of tensile cracking of concrete suggests $P_u/\phi_c P_n = 0.3$ as a conservative limit. It is assumed that when $P_u/\phi_c P_n$ is less than 0.3, the nominal flexural strength is reduced below that indicated by plastic stress distribution on the composite cross section unless the transfer of shear from the concrete to the steel is provided for by shear connectors.

SECTION C9.5
SHEAR CONNECTORS

C9.5.2 **Horizontal Shear Force.** Composite beams in which the longitudinal spacing of shear connectors was varied according to the intensity of the static shear, and duplicate beams in which the connectors were uniformly spaced, exhibited the same ultimate strength and the same amount of deflection at normal working loads. Only a slight deformation in the concrete and the more heavily stressed connectors is needed to redistribute the horizontal shear to other less heavily stressed connectors. The important consideration is that the total number of connectors be sufficient to develop the shear V_h on either side of the point of maximum moment. The provisions of the SBC 306 are based upon this concept of composite action.

In computing the design flexural strength at points of maximum negative bending, reinforcement parallel to the steel beam within the effective width of the slab may be included, provided such reinforcement is properly anchored beyond the region of negative moment. However, enough shear connectors are required to transfer the ultimate tensile force in the reinforcement, from the slab to the steel beam.

C9.5.3 **Strength of Stud Shear Connectors.** The SBC 306 does not specify a resistance factor for shear connector strength. The resistance factor for the flexural strength of a composite beam accounts for all sources of variability, including those associated with the shear connectors.

C9.5.6 **Shear Connector Placement and Spacing.** Uniform spacing of shear connectors is permitted except in the presence of heavy concentrated loads.

Studs not located directly over the web of a beam tend to tear out of a thin flange before attaining full shear-resisting capacity. To guard against this contingency, the size of a stud not located over the beam web is limited to 2½ times the flange thickness (Goble, 1968).

The minimum spacing of connectors along the length of the beam, in both flat soffit concrete slabs and in formed steel deck with ribs parallel to the beam, is

six diameters; this spacing reflects development of shear planes in the concrete slab (Ollgaard et al., 1971). Since most test data are based on the minimum transverse spacing of four diameters, this transverse spacing was set as the minimum permitted. If the steel beam flange is narrow, this spacing requirement may be achieved by staggering the studs with a minimum transverse spacing of three diameters between the staggered rows of studs. The reduction in connector capacity in the ribs of formed steel decks is provided by the factor $0.85/\sqrt{N_r}$, which accounts for the reduced capacity of multiple connectors, including the effect of spacing. When deck ribs are parallel to the beam and the design requires more studs than can be placed in the rib, the deck may be split so that adequate spacing is available for stud installation. Figure C9.5-1 shows possible connector arrangements.

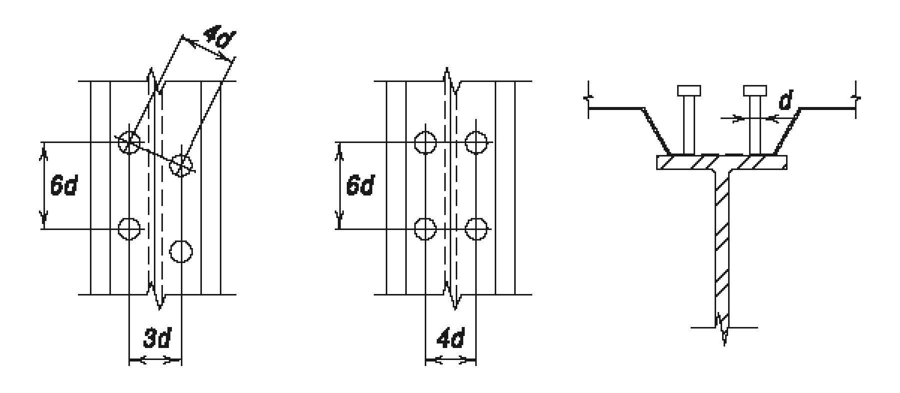


Figure. C9.5-1 Shear connector arrangements.

SECTION C9.6
SPECIAL CASES

Tests are required for construction that falls outside the limits given in the Specification. Different types of shear connectors may require different spacing and other detailing than stud and channel connectors.

CHAPTER 10
CONNECTIONS, JOINTS AND FASTENERS

SECTION C10.1
GENERAL PROVISIONS

C10.1.5 **Splices in Heavy Sections.** Solidified but still-hot filler metal contracts significantly as it cools to ambient temperature. Shrinkage of large welds between elements which are not free to move to accommodate the shrinkage causes strains in the material adjacent to the weld that can exceed the yield point strain. In thick material the weld shrinkage is restrained in the thickness direction, as well as in the width and length directions, causing triaxial stresses to develop that may inhibit the ability of ductile steel to deform in a ductile manner. Under these conditions, the possibility of brittle fracture increases.

When splicing ASTM A6/A6M Group 4 and 5 and equivalent rolled sections or heavy welded built-up members, the potentially harmful weld shrinkage strains can be avoided by using bolted splices or fillet-welded lap splices or splices that combine a welded and bolted detail (see Figure C10.1-1). Details and techniques that perform well for materials of modest thickness usually must be changed or supplemented by more demanding requirements when welding thick material. Also, the provisions of the *Structural Welding Code*, AWS D1.1, are minimum requirements that apply to most structural welding situations; however, when designing and fabricating welded splices of ASTM A6/A6M Group 4 and 5 and equivalent shapes and similar built-up cross sections, special consideration must be given to all aspects of the welded splice detail.

- Notch-toughness requirements should be specified for tension members. See Commentary Section C1.3.
- Generously sized weld access holes, Figure C10.1-2, are required to provide increased relief from concentrated weld shrinkage strains, to avoid close juncture of welds in orthogonal directions, and to provide adequate clearance for the exercise of high quality workmanship in hole preparation, welding, and ease of inspection.
- Preheating for thermal cutting is required to minimize the formation of a hard surface layer.
- Grinding to bright metal and inspection using magnetic particle or dye-penetrant methods is required to remove the hard surface layer and to assure smooth transitions free of notches or cracks.

In addition to tension splices of truss chord members and tension flanges of flexural members, other joints fabricated of heavy sections subject to tension should be given special consideration during design and fabrication.

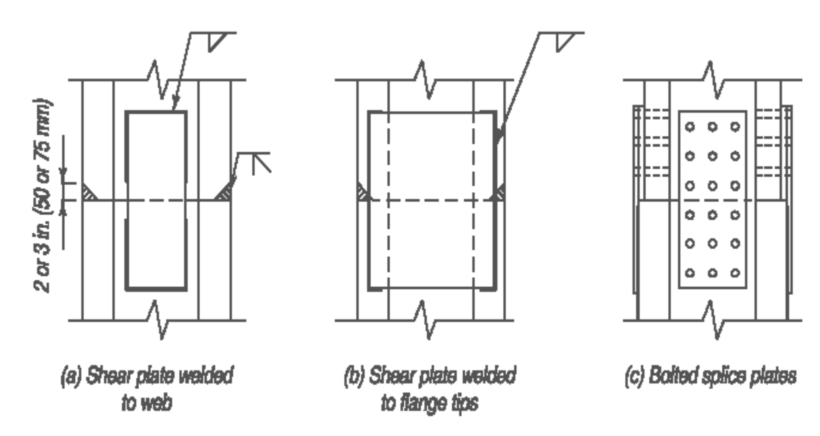
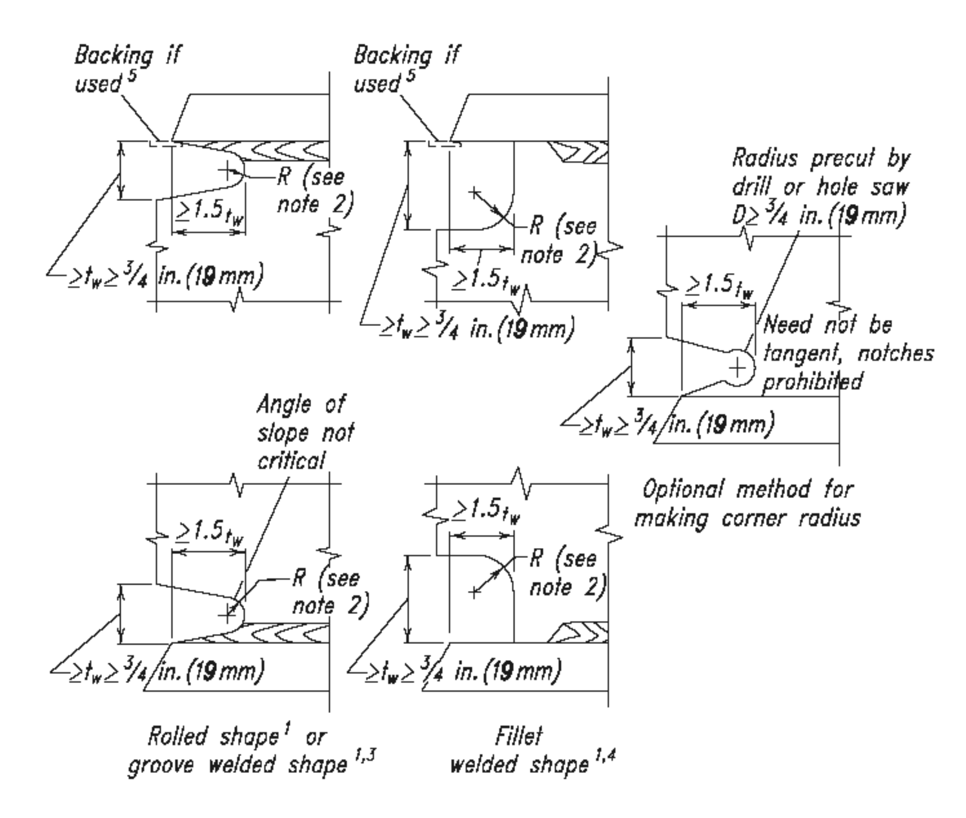


Figure. C10.1-1. Alternative splices that minimize weld restraint tensile stresses.



Notes:

1. *For ASTM A6 Group 4 and 5 and equivalent shapes and welded built-up shapes with plate thickness more than 50 mm, preheat to 65° C prior to thermal cutting, grind and inspect thermally cut edges of access hole using magnetic particle or dye penetration methods prior to making web and flange splice groove welds.*
2. *Radius shall provide smooth notch-free transition; $R \geq 10$ mm (typical 13 mm).*
3. *Access opening made after welding web to flange.*
4. *Access opening made before welding web to flange.*
5. *These are typical details for joints welded from one side against steel backing. Alternative joint designs should be considered.*

Figure. C10.1-2. Weld access hole geometry

C10.1.8 Placement of Welds and Bolts. The fatigue life of eccentrically loaded welded angles has been shown to be very short (Kloppel and Seeger, 1964). Notches at the roots of fillet welds are harmful when alternating tensile stresses are normal to the axis of the weld, as could occur due to bending when axial cyclic loading is applied to angles with end welds not balanced about the neutral axis. Accordingly, balanced welds are indicated when such members are subjected to cyclic loading (see Figure C10.1-3).

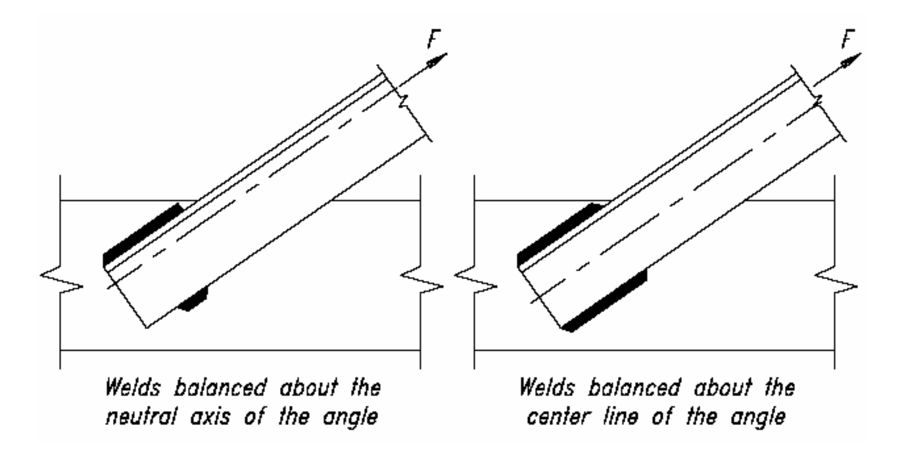


Figure C10.1-3. Balanced welds.

C10.1.9 Bolts in Combination with Welds. The sharing of load between welds and A307 bolts or high-strength bolts in a bearing-type connection is not recommended. For similar reasons, A307 bolts and rivets should not be assumed to share loads in a single group of fasteners.

For high-strength bolts in slip-critical connections to share the load with welds it is advisable to fully tension the bolts before the weld is made. If the weld is placed first, angular distortion from the heat of the weld might prevent the faying action required for development of the slip-critical force. When the bolts are fully tensioned before the weld is made, the slip-critical bolts and the weld may be assumed to share the load on a common shear plane (Kulak, Fisher, and Struik, 1987). The heat of welding near bolts will not alter the mechanical properties of the bolts.

In making alterations to existing structures, it is assumed that whatever slip is likely to occur in high-strength bolted bearing-type connections or riveted connections will have already taken place. Hence, in such cases the use of welding to resist all stresses, other than those produced by existing dead load present at the time of making the alteration, is permitted.

It should be noted that combinations of fasteners as defined herein does not refer to connections such as shear plates for beam-to-column connections which are welded to the column and bolted to the beam flange or web (Kulak et al., 1987) and other comparable connections.

C10.1.10 High-Strength Bolts in Combination with Rivets. When high-strength bolts are used in combination with rivets, the ductility of the rivets permits the direct addition of the strengths of both fastener types.

SECTION C10.2
WELDS

C10.2.1 **Groove Welds.** The engineer preparing contract design drawings cannot specify the depth of groove without knowing the welding process and the position of welding. Accordingly, only the effective throat for partial-joint-penetration groove welds should be specified on design drawings, allowing the fabricator to produce this effective throat with his own choice of welding process and position. The weld reinforcement is not used in determining the effective throat thickness of a groove weld (see Table 10.2-1).

C10.2.2 **Fillet Welds.**

C10.2.2.1 **Effective Area**. The effective throat of a fillet weld is based upon the root of the joint and the face of the diagrammatic weld; hence this definition gives no credit for weld penetration or reinforcement at the weld face. If the fillet weld is made by the submerged arc welding process, some credit for penetration is made. If the leg size of the resulting fillet weld exceeds 10 mm, then 3 mm is added to the theoretical throat. This increased weld throat is allowed because the submerged arc process produces deep penetration of welds of consistent quality. However, it is necessary to run a short length of fillet weld to be assured that this increased penetration is obtained. In practice, this is usually done initially by cross-sectioning the runoff plates of the joint. Once this is done, no further testing is required, as long as the welding procedure is not changed.

C10.2.2.2 **Limitations**. Table 10.2-4 provides a minimum size of fillet weld for a given thickness of the thicker part joined.

The requirements are not based upon strength considerations, but upon the quench effect of thick material on small welds. Very rapid cooling of weld metal may result in a loss of ductility. Further, the restraint to weld metal shrinkage provided by thick material may result in weld cracking. Because 8 mm fillet weld is the largest that can be deposited in a single pass by SMAW process, 8 mm applies to all material 19 mm and greater in thickness, but minimum preheat and inter-pass temperature are required by AWS D1.1 (See Table 10.2-4). Both the design engineer and the shop welder must be governed by the requirements.

Table 10.2-3 gives the minimum effective throat of a partial-joint-penetration groove weld. Notice that Table 10.2-3 for partial-joint-penetration groove welds goes up to a plate thickness of over 150 mm and a minimum weld throat of 16 mm, whereas, for fillet welds Table 10.2-4 goes up to a plate thickness of over 19 mm and a minimum leg size of fillet weld of only 8 mm. The additional thickness for partial-joint-penetration groove welds is to provide for reasonable proportionality between weld and material thickness.

For plates of 6 mm or more in thickness, it is necessary that the inspector be able to identify the edge of the plate to position the weld gage. This is assured if the weld is kept back at least 2 mm from the edge, as shown in Figure C10.2-1.

Where longitudinal fillet welds are used alone in a connection (see Figure C10.2-2), Section 10.2.2.2 requires the length of each weld to be at least equal to the width of the connecting material because of shear lag (Freeman, 1930).

By providing a minimum lap of five times the thickness of the thinner part of a lap joint, the resulting rotation of the joint when pulled will not be excessive, as shown

in Figure C10.2-3. Fillet welded lap joints under tension tend to open and apply a tearing action at the root of the weld as shown in Figure C10.2-4(b), unless restrained by a force F as shown in Figure C10.2-4(a).

End returns are not essential for developing the capacity of fillet welded connections and have a negligible effect on their strength. Their use has been encouraged to insure that the weld size is maintained over the length of the weld, to enhance the fatigue resistance of cyclically loaded flexible end connections, and to increase the plastic deformation capability of such connections.

The weld capacity database on which the SBC 306 was developed had no end returns. This includes the study by Higgins and Preece (1968), seat angle tests by Lyse and Schreiner (1935), the seat and top angle tests by Lyse and Gibson (1937), beam webs welded directly to column or girder by fillet welds by Johnston and Deits (1942), and the eccentrically loaded welded connections reported by Butler, Pal, and Kulak (1972). Hence, the current design-resistance values and joint-capacity models do not require end returns when the required weld size is provided. Johnston and Green (1940) noted that movement consistent with the design assumption of no end restraint (i.e., joint flexibility) was enhanced without end returns. They also verified that greater plastic deformation of the connection was achieved when end returns existed, although the strength was not significantly different.

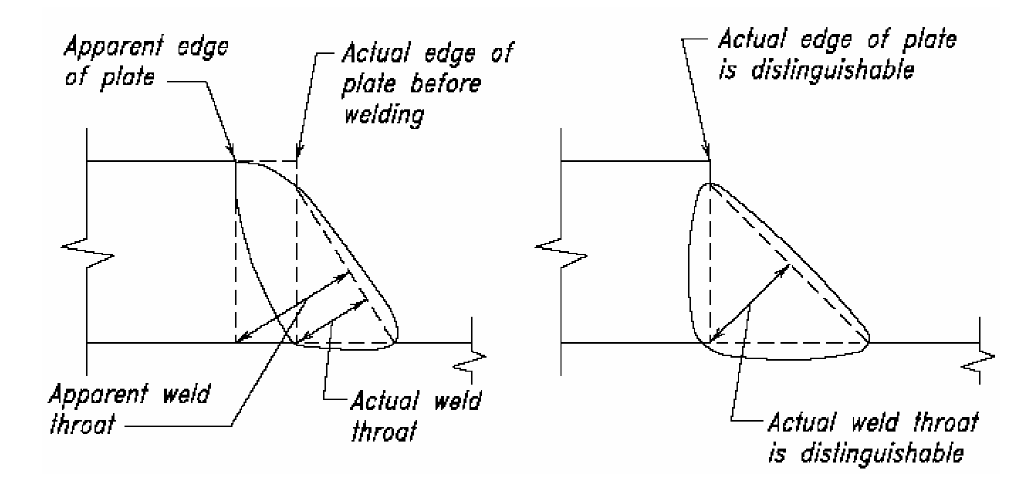


Figure. C10.2-1. Identification of plate edge.

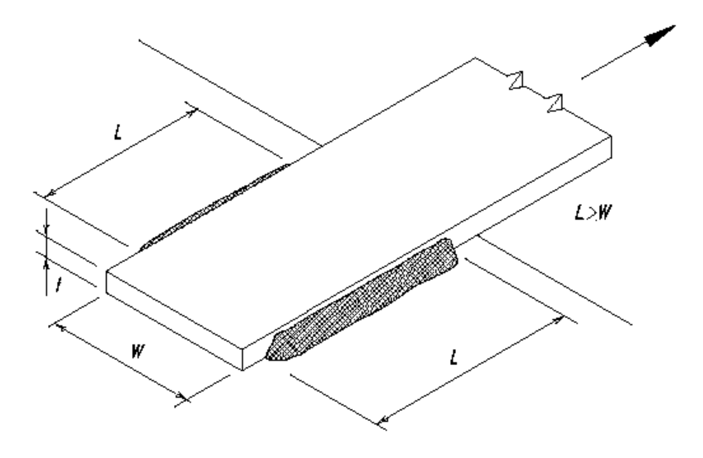


Figure. C10.2-2. Longitudinal fillet welds.

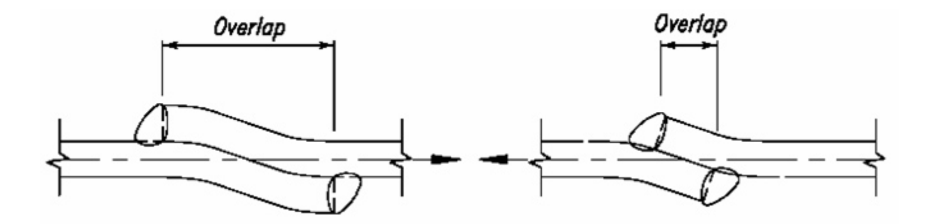


Figure. C10.2-3. Minimum lap.

When longitudinal fillet welds parallel to the stress are used to transmit the load to the end of an axially loaded member, the welds are termed “end loaded”. Typical examples of such welds would include, but are not necessarily limited to, longitudinally welded lap joints at the end of axially loaded members, welds attaching bearing stiffeners, and similar cases. Typical examples of longitudinally loaded fillet welds which are not considered end loaded include, but are not limited to, welds that connect plates or shapes to form built-up cross sections in which the shear force is applied to each increment of length of weld stress depending upon the distribution of shear load along the length of the member, welds attaching beam web connection angles and shear plates because the flow of shear force from the beam or girder web to the weld is essentially uniform throughout the weld length, that is, the weld is not end-loaded despite the fact that it is loaded parallel to the weld axis. Neither does the reduction factor apply to welds attaching stiffeners to webs because the stiffeners and welds are not subject to calculated axial stress but merely serve to keep the web flat.

The distribution of stress along the length of end loaded fillet welds is far from uniform and is dependent upon complex relationships between the stiffness of the longitudinal fillet weld relative to the stiffness of the connected materials. Beyond some length, it is non-conservative to assume that the average stress over the total length of the weld may be taken as equal to the full design strength. Experience has shown that when the length of the weld is equal to approximately 100 times the weld size or less, it is reasonable to assume the effective length is equal to the actual length. For weld lengths greater than 100 times the weld size, the effective length should be taken less than the actual length. The reduction coefficient, R, provided in Section 10.2.2.2 is the equivalent of Eurocode 3, which is a simplified approximation to exponential formulas developed by finite element studies and tests preformed in Europe over many years. The criterion is based upon combined consideration of nominal strength for fillet welds with leg size less than 6 mm and upon a judgment based serviceability limit of slightly less than 1 mm displacement at the end of the weld for welds with leg size 6 mm and larger. Mathematically, the application of the β factor implies that the minimum strength of an end-loaded weld is achieved when the length is approximately 300 times the leg size. Because it is illogical to conclude that the total strength of a weld longer than 300 times the weld size would be less than a shorter weld, the length reduction coefficient is taken as 0.6 when the weld length is greater than 300 times the leg size.

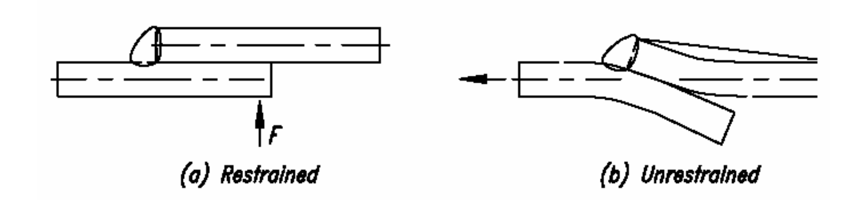


Figure. C10.2-4. Restraint of lap joints.

Fillet weld terminations do not affect the strength or serviceability of connections in most cases. However, in certain cases, the disposition of welds affect the planned function of connections, and notches may affect the static strength and/or the resistance to crack initiation if cyclic loads of sufficient magnitude and frequency occur. For these cases, limitations are specified to assure desired performance.

(a) At lapped joints where one part extends beyond the end or edge of the part to which it is welded and if the parts are subject to calculated tensile stress at the start of the overlap, it is important that the weld terminate a short distance from the stressed edge. For one typical example, the lap joint between the tee chord and the web members of a truss, the weld should not extend to the edge of the tee stem. See Figure C10.2-5. The best technique to avoid inadvertent notches at this critical location is to strike the welding arc at a point slightly back from the edge and proceed with welding in the direction away from the edge. See Figure C10.2-6. On the other hand, where framing angles extend beyond the end of the beam web to which they are welded, the free end of the beam web is subject to zero stress; thus, it is permissible for the fillet weld to extend continuously across the top end, along the side and along the bottom end of the angle to the extreme end of the beam. See Figure C10.2-7.

(b) For connections which are subject to maximum stress at the weld termination due to cyclic forces and/or moments of sufficient magnitude and frequency to initiate cracks emanating from unfilled start or stop craters or other discontinuities, at the end of the weld must be protected by boxing or returns. If the bracket is a plate projecting from the face of a support, extra care must be exercised in the deposition of the boxing weld across the thickness of the plate to assure that a fillet free of notches is provided.

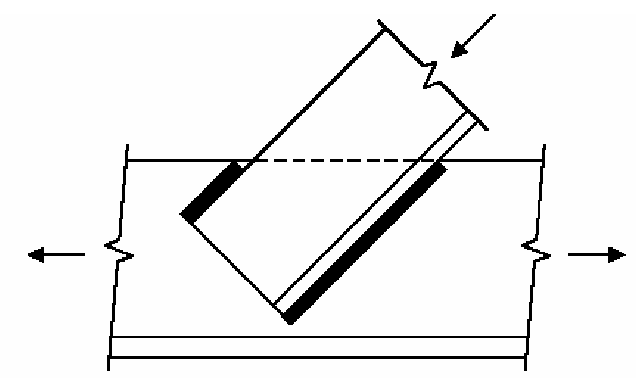


Figure. C10.2-5. Fillet welds near tension edges

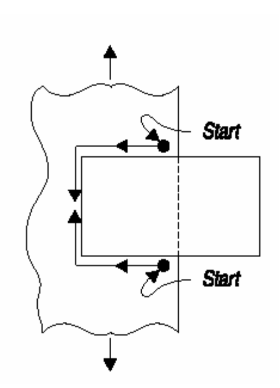


Figure. C10.2-6. Suggested direction of welding travel to avoid notches.

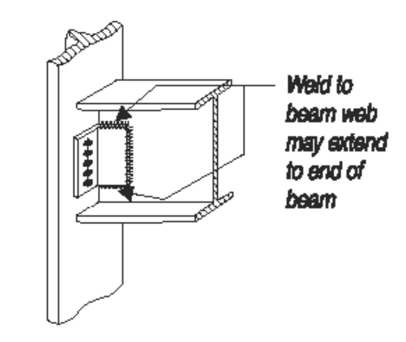


Figure. C10.2-7. Fillet weld details on framing angles.

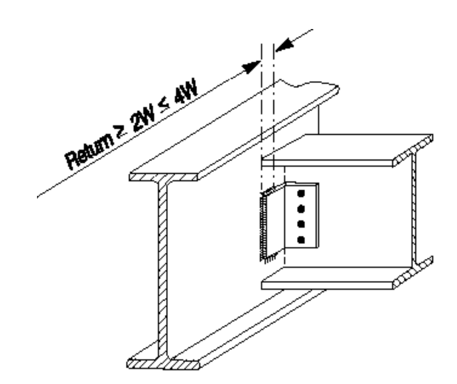


Figure. C10.2-8. Flexible connection returns optional unless subject to fatigue.

(c) For connections such as framing angles and simple end plates which are assumed in design of the structure to be flexible connections, the top and bottom edges of the outstanding legs must be left unwelded over a substantial portion of their length in order to assure flexibility of the connection. Research tests (Johnston and Green, 1940) have shown that the static strength of the connection is the same with or without end returns; therefore the use of returns is optional, but if used, their length must be restricted to not more than four times the weld size, see Figure C10.2-8.

(d) Experience has shown that when ends of intermediate transverse stiffeners on the webs of plate girders are not welded to the flanges (the usual practice), small torsional distortions of the flange which occur near shipping bearing points in the normal course of shipping by rail or truck may cause

high out-of-plane bending stresses (yield point) and fatigue cracking at the toe of the web-to-flange welds. This has been observed even with closely fitted stiffeners. The intensity of these out-of-plane stresses may be effectively limited and cracking prevented if “breathing room” is provided by terminating web-to-flange welds. The unwelded distance should not exceed six times the web thickness to assure that column buckling of the web within the unwelded length does not occur.

(e) For fillet welds which occur on opposite sides of a common plane, it is not possible to deposit a weld continuously around the corner from one side to the other without causing a gouge in the corner of the parts joined; therefore the welds must be interrupted at the corner. See Figure C10.2-9.

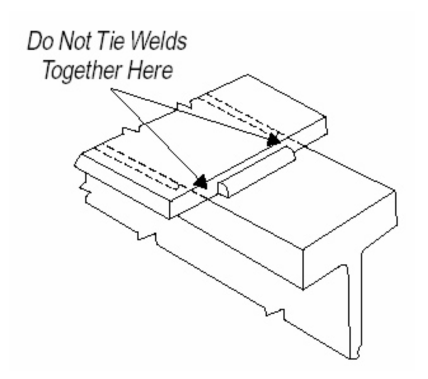


Figure. C10.2-9. Details for fillet welds which occur on opposite sides of a common plane.

C10.2.4 **Design Strength.** The strength of welds is governed by the strength of either the base material or the deposited weld metal. Table 10.2-5 contains the resistance factors and nominal weld strengths, as well as a number of limitations.

It should be noted that in Table 10.2-5 the nominal strength of fillet welds is determined from the effective throat area, whereas the strength of the connected parts is governed by their respective thicknesses. Figure C10.2-10 illustrates the shear planes for fillet welds and base material:

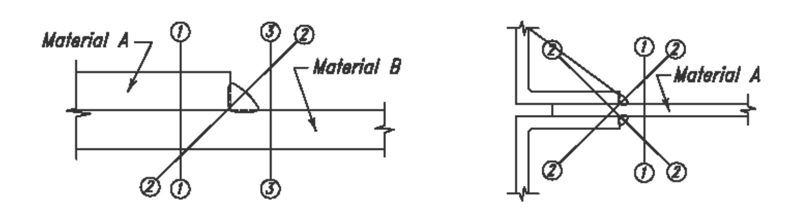


Figure. C10.2-10. Shear planes for fillet welds loaded in longitudinal shear

(a) Plane 1-1, in which the resistance is governed by the shear strength for material A.

(b) Plane 2-2, in which the resistance is governed by the shear strength of the weld metal.

(c) Plane 3-3, in which the resistance is governed by the shear strength of the material B.

The resistance of the welded joint is the lowest of the resistance calculated in each plane of shear transfer. Note that planes 1-1 and 3-3 are positioned away from the fusion areas between the weld and the base material. Tests have demonstrated that the stress on this fusion area is not critical in determining the shear strength of fillet welds (Preece, 1968).

The shear planes for plug and partial-joint-penetration groove welds are shown in Figure C10.2-12 for the weld and base metal. Generally the base metal will govern the shear strength.

When weld groups are loaded in shear by an external load that does not act through the center of gravity of the group, the load is eccentric and will tend to cause a relative rotation and translation between the parts connected by the weld. The point about which rotation tends to take place is called the instantaneous center of rotation. Its location is dependent upon the load eccentricity, geometry of the weld group, and deformation of the weld at different angles of the resultant elemental force relative to the weld axis.

The individual resistance force of each unit weld element can be assumed to act on a line perpendicular to a ray passing through the instantaneous center and that element's location (see Figure C10.2-11).

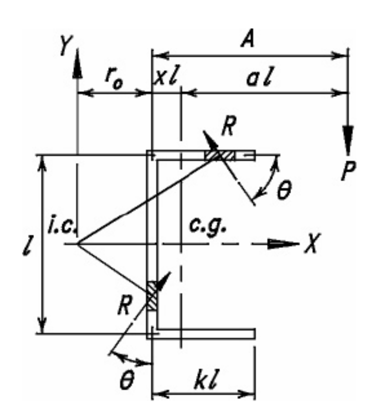


Figure. C10.2-11. Weld element nomenclature.

C10.2.5 **Combination of Welds**. This method of adding weld strengths does not apply to a welded joint using a partial-joint-penetration single bevel groove weld with a superimposed fillet weld. In this case, the effective throat of the combined joint must be determined and the design strength based upon this throat area.

C10.2.6 **Weld Metal Requirements**. Applied and residual stresses and geometrical discontinuities from back-up bars with associated notch effects contribute to sensitivity to fracture. Some weld metals in combination with certain procedures result in welds with low notch toughness. The Specification requires a minimum specified toughness for weld metals in those joints that are subject to more significant applied stresses and toughness demands.

The level of toughness required was selected as one level more conservative than the base metal requirement for Group 4 and 5 and equivalent shapes. Research continues on this subject.

C10.2.7 Mixed Weld Metal. Problems can occur when incompatible weld metals are used in combination and notch-tough composite weld metal is required. For instance, tack welds deposited using a self-shielded process with aluminum deoxidizers in the electrodes and subsequently covered by SAW weld passes can result in composite weld metal with low notch-toughness, despite the fact that each process by itself could provide notch-tough weld metal.

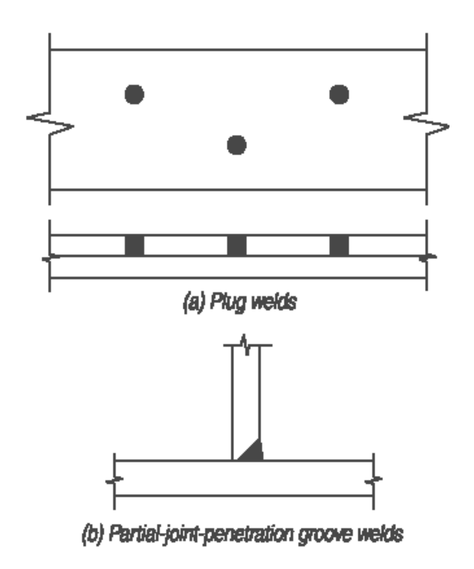


Figure. C10.2-12. Shear planes for plug and partial-joint-penetration groove welds.

SECTION C10.3
BOLTS AND THREADED PARTS

C10.3.1 High-Strength Bolts. In general, the use of high-strength bolts is required to conform to the provisions of the *Load and Resistance Factor Design Specification for Structural Joints Using ASTM A325 or A490 Bolts* (RCSC, 1994) as approved by the Research Council on Structural Connections.

Occasionally the need arises for the use of high-strength bolts of diameters and lengths in excess of those available for A325 or A325M and A490 or A490M bolts, as for example, anchor rods for fastening machine bases. For this situation Section 1.3.3 permits the use of A449 bolts and A354 threaded rods.

With this edition of the Specification snug-tightened installation is permitted for static applications involving ASTM A325 or A325M bolts (only) in tension or combined shear and tension.

There are practical cases in the design of structures where slip of the connection is desirable in order to allow for expansion and contraction of a joint in a controlled manner. Regardless of whether force transfer is required in the directions normal to the slip direction, the nuts should be hand-tightened with a spud wrench and then backed off one-quarter turn. Furthermore, it is advisable to deform the bolt threads or use a locking nut or jamb nut to insure that the nut does not back off under service conditions. Thread deformation is commonly accomplished with a cold

chisel and hammer applied at one location. Note that tack-welding of the nut to the bolt threads is discouraged.

C10.3.2 **Size and Use of Holes.** To provide some latitude for adjustment in plumbing up a frame during erection, three types of enlarged holes are permitted, subject to the approval of the designer. The nominal maximum sizes of these holes are given in Table 10.3-3. The use of these enlarged holes is restricted to connections assembled with bolts and is subject to the provisions of Sections 10.3.3 and 10.3.4.

C10.3.3 **Minimum Spacing.** The maximum factored strength R_n at a bolt or rivet hole in bearing requires that the distance between the centerline of the first fastener and the edge of a plate toward which the force is directed should not be less than $1\frac{1}{2}d$ where d is the fastener diameter (Kulak et al., 1987). By similar reasoning the distance measured in the line of force, from the centerline of any fastener to the nearest edge of an adjacent hole, should not be less than $3d$, to ensure maximum design strength in bearing. Plotting of numerous test results indicates that the critical bearing strength is directly proportional to the above defined distances up to a maximum value of $3d$, above which no additional bearing strength is achieved (Kulak et al., 1987). Table 10.3-6 lists the increments that must be added to adjust the spacing upward to compensate for an increase in hole dimension parallel to the line of force. Section 10.3.10 gives the bearing strength criteria as a function of spacing.

C10.3.4 **Minimum Edge Distance.** Critical bearing stress is a function of the material tensile strength, the spacing of fasteners, and the distance from the edge of the part to the center line of the nearest fastener. Tests have shown (Kulak et al., 1987) that a linear relationship exists between the ratio of critical bearing stress to tensile strength (of the connected material) and the ratio of fastener spacing (in the line of force) to fastener diameter. The following equation affords a good lower bound to published test data for single-fastener connections with standard holes, and is conservative for adequately spaced multi-fastener connections:

$$F_{pcr} / F_u = l_e / d \qquad \text{(C10.3-1)}$$

where

F_{pcr} = critical bearing stress, MPa

F_u = tensile strength of the connected material, MPa

l_e = distance, along a line of transmitted force, from the center of a fastener to the nearest edge of an adjacent fastener or to the free edge of a connected part (in the direction of stress), mm

d = diameter of fastener, mm

C10.3.5 **Maximum Spacing and Edge Distance.** Limiting the edge distance to not more than 12 times the thickness of an outside connected part, but not more than 150 mm, is intended to provide for the exclusion of moisture in the event of paint failure, thus preventing corrosion between the parts which might accumulate and

force these parts to separate. More restrictive limitations are required for connected parts of unpainted weathering steel exposed to atmospheric corrosion.

C10.3.6 **Design Tension or Shear Strength.** Tension loading of fasteners is usually accompanied by some bending due to the deformation of the connected parts. Hence, the resistance factor ϕ, by which R_n is multiplied to obtain the design tensile strength of fasteners, is relatively low. The nominal tensile strength values in Table 10.3-2 were obtained from the equation

$$R_n = 0.75 A_b F_u \qquad \text{(C10.3-2)}$$

This tensile strength given by Equation C10.3-2 is independent of whether the bolt was initially installed pretensioned or snug-tightened.

In connections consisting of only a few fasteners, the effects of strain on the shear in bearing fasteners is negligible (Kulak et al., 1987 and Fisher et al., 1978). In longer joints, the differential strain produces an uneven distribution between fasteners (those near the end taking a disproportionate part of the total load), so that the maximum strength per fastener is reduced. The ASD-based Specifications permits connections up to 1270 mm in length without a reduction in maximum shear stress. With this in mind the resistance factor ϕ for shear in bearing-type connections has been selected to accommodate the same range of connections.

The values of nominal shear strength in Table 10.3-2 were obtained from the equation

$$R_n / mA_b = 0.50\, F_u \qquad \text{(C10.3-3)}$$

when threads are excluded from the shear planes and

$$R_n / mA_b = 0.40\, F_u \qquad \text{(C10.3-4)}$$

when threads are not excluded from the shear plane, where m is the number of shear planes (Kulak et al., 1987). While developed for bolted connections, the equations were also conservatively applied to threaded parts and rivets. The value given for A307 bolts was obtained from Equation C10.3-4 but is specified for all cases regardless of the position of threads. For A325 or A325M bolts, no distinction is made between small and large diameters, even though the minimum tensile strength F_u is lower for bolts with diameters in excess of one inch. It was felt that such a refinement of design was not justified, particularly in view of the low resistance factor ϕ, the increasing ratio of tensile area to gross area, and other compensating factors.

C10.3.7 **Combined Tension and Shear in Bearing-Type Connections.** Tests have shown that the strength of bearing fasteners subject to combined shear and tension resulting from externally applied forces can be closely defined by an ellipse (Kulak et al., 1987). Such a curve can be replaced, with only minor deviations, by three straight lines as shown in Figure C10.3-1. This latter representation offers the advantage that no modification of either type stress is required in the presence

of fairly large magnitudes of the other type. This linear representation was adopted for Table 10.3-5, giving a limiting tensile stress F_t as a function of the shearing stress f_v for bearing-type connections. Following a change in the 1994 RCSC *LRFD Specification for Structural Joints Using ASTM A325 or A490 Bolts,* the coefficients in the equations in Table 10.3-5 have been modified for consistency (Carter, Tide, and Yura, 1997).

C10.3.8 High-Strength Bolts in Slip-Critical Connections. Connections classified as slip-critical include those cases where slip could theoretically exceed an amount deemed by the Engineer of Record to affect the suitability for service of the structure by excessive distortion or reduction in strength or stability, even though the nominal strength of the connection may be adequate. Also included are those cases where slip of any magnitude must be prevented, for example, joints subject to fatigue, connectors between elements of built-up members at their ends (Sections 4.2 and 5.4), and bolts in combination with welds (Section 10.1.9).

The onset of slipping in a high-strength bolted, slip-critical connection is not an indication that the maximum strength of the connection has been reached. Its occurrence may be only a serviceability limit state. The design check for slip resistance can be made at two different load levels, factored loads (Sections 10.3.8.1 and 10.3.9.1) and service loads (not included here). The nominal slip resistances r_{str} and F_yA_b to be used with factored loads and service loads, respectively, are based on two different design concepts. The slip resistance r_{str} with factored loads is the mean resistance per bolt, which is a function of the mean slip coefficient and the clamping force. The 1.13 factor in (Equation 10.3-1) accounts for the expected 13 percent increase above the minimum specified preload provided by calibrated wrench tightening procedures. This was used to represent typical installations. The factored load resistance r_{str} uses the β reliability index approach that is used for the other design checks such as tension and bearing. The service load approach uses a probability of slip concept that implies a 90 percent reliability that slip will not occur if the calibrated wrench method of bolt installation is used.

The Engineer of Record must make the determination to use factored loads, service loads, or both in checking the slip resistance of a slip-critical connection. The following commentary is provided as guidance and an indication of the intent of the Specification.

In the case of slip-critical connections with three or more bolts in holes with only a small clearance, such as standard holes and slotted holes loaded transversely to the axis of the slot, the freedom to slip does not generally exist because one or more bolts are in bearing even before load is applied due to normal fabrication tolerances and erection procedures. If connections with standard holes have only one or two bolts in the direction of the applied force, a small slip may occur. In this case, slip-critical connections subjected to vibration or wind should be checked for slip at service-load levels. In built-up compression members, such as double-angle struts in trusses, a small slip in the end connections can significantly reduce the strength of the compression member so the slip-critical end connection should be checked for slip at the factored-load level, whether or not a slip-critical connection is required by a serviceability requirement.

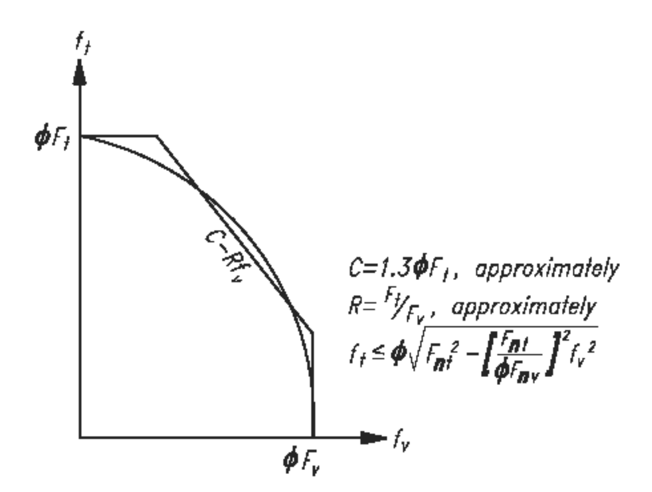


Figure. C10.3-1. Three straight line approximation.

In connections with long slots that are parallel to the direction of the applied load, slip of the connection prior to attainment of the factored load might be large enough to alter the usual assumption of analysis that the undeformed structure can be used to obtain the internal forces. The SBC 306 allows the designer two alternatives in this case. If the connection is designed so that it will not slip under the effects of service loads, then the effect of the factored loads acting on the deformed structure (deformed by the maximum amount of slip in the long slots at all locations) must be included in the structural analysis. Alternatively, the connection can be designed so that it will not slip at loads up to the factored load level.

Joints subjected to full reverse cyclical loading are clearly slip-critical joints since slip would permit back and forth movement of the joint and early fatigue. However, for joints subjected to pulsating load that does not involve reversal of direction, proper fatigue design could be provided either as a slip-critical joint on the basis of stress on the gross section, or as a non-slip-critical joint on the basis of stress on the net section. Because fatigue results from repeated application of the service load rather than the overload load, design should be based upon service-load criteria.

For high-strength bolts in combination with welds in statically loaded conditions and considering new work only, the nominal strength may be taken as the sum of the slip resistances provided by the bolts and the shear resistance of the welds. Section 10.1.9 requires that the slip resistance be determined at factored load levels. If one type of connector is already loaded when the second type of connector is introduced, the nominal strength cannot by obtained by adding the two resistances. The Guide (Kulak et al., 1987) should be consulted in these cases.

Slip of slip-critical connections is likely to occur at approximately 1.4 to 1.5 times the service loads. For standard holes, oversized holes, and short-slotted holes the connection is designed at factored loads (Section 10.3.8.1). The nominal loads and ϕ factors have been adjusted accordingly. The number of connectors will be essentially the same for the two procedures because they have been calibrated to give similar results. Slight differences will occur because of variation in the ratio of live load to dead load.

In connections containing long slots that are parallel to the direction of the applied load, slip of the connection prior to attainment of the factored load might be large enough to alter the usual assumption of analysis that the undeformed structure can

be used to obtain the internal forces. To guard against this occurring, the design slip resistance is further reduced by setting ϕ to 0.60 in conjunction with factored loads.

While the possibility of a slip-critical connection slipping into bearing under anticipated service conditions is small, such connections must comply with the provisions of Section 10.3.10 in order to prevent connection failure at the maximum load condition.

C10.3.10 **Bearing Strength at Bolt Holes.** Provisions for bearing strength of pins differ from those for bearing strength of bolts; refer to Section 10.8.

Bearing values are provided as a measure of the strength of the material upon which a bolt bears, not as a protection to the fastener, which needs no such protection. Accordingly, the same bearing value applies to all joints assembled by bolts, regardless of fastener shear strength or the presence or absence of threads in the bearing area.

Material bearing strength may be limited either by bearing deformation of the hole or by block shear rupture of the material upon which the bolt bears. Recent testing by Kim and Yura (1996) and Lewis and Zwerneman (1996) has confirmed the bearing strength provisions for the former case wherein the nominal bearing strength R_n is equal to $CdtF_u$ and C is 2.4, 3.0, or 2.0 depending upon hole type and/or acceptability of hole ovalization at ultimate load as indicated in LRFD Specification Section 10.3.10. However, this same research indicated the need for more accurate bearing strength provisions when block-shear-rupture-type failure would control. Appropriate equations for bearing strength as a function of clear distance L_c are therefore provided and this formulation is consistent with that adopted by RCSC in the Load and Resistance Factor Design Specification for *Structural Joints Using ASTM A325 or A490 Bolts* (RCSC, 1994).

Additionally, to simplify and generalize such bearing strength calculations, the current provisions have been based upon a clear-distance formulation. Previous provisions utilized edge distances and bolt spacings measured to hole centerlines with adjustment factors to account for varying hole type and orientation, as well as minimum edge distance requirements.

C10.3.11 **Long Grips.** Provisions requiring a decrease in calculated stress for A307 bolts having long grips (by arbitrarily increasing the required number in proportion to the grip length) are not required for high-strength bolts. Tests (Bendigo, Hansen, and Rumpf, 1963) have demonstrated that the ultimate shearing strength of high-strength bolts having a grip of eight or nine diameters is no less than that of similar bolts with much shorter grips.

SECTION C10.4 DESIGN RUPTURE STRENGTH

Tests (Birkemoe and Gilmor, 1978) on coped beams indicated that a tearing failure mode (rupture) can occur along the perimeter of the bolt holes as shown in Figure C10.4-1. This block shear mode combines tensile strength on one plane and shear strength on a perpendicular plane. The failure path is defined by the center lines of

the bolt holes. The block shear failure mode is not limited to the coped ends of beams. Other examples are shown in Figure C10.4-1 and C10.4-2.

The block shear failure mode should also be checked around the periphery of welded connections. Welded connection block shear is determined using $\phi = 0.75$ in conjunction with the area of both the fracture and yielding planes (Yura, 1988).

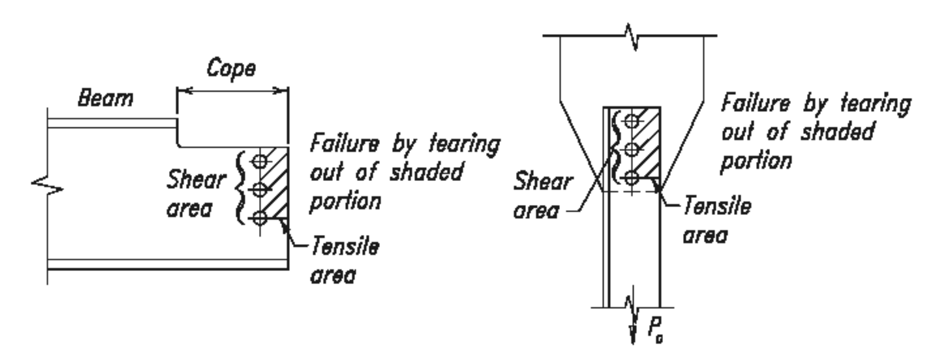


Figure. C10.4-1. Failure for block shear rupture limit state.

The LRFD Specification has adopted a conservative model to predict block shear strength. Test results suggest that it is reasonable to add the yield strength on one plane to the rupture strength of the perpendicular plane (Ricles and Yura, 1983, and Hardash and Bjorhovde, 1985). Therefore, two possible block shear strengths can be calculated; rupture strength F_u on the net tensile section along with shear yielding $0.6\ F_y$ on the gross section on the shear plane(s), or rupture $0.6\ F_u$ on the net shear area(s) combined with yielding F_y on the gross tensile area. This is the basis of Equations 10.4-1 and 10.4-2.

These equations are consistent with the philosophy in Chapter 4 for tension members, where gross area is used for the limit state of yielding and net area is used for rupture. The controlling equation is the one that produces the larger rupture force.

This can be explained by the two extreme examples given in Figure C10.4-2. In Case (a), the total force is resisted primarily by shear, so shear rupture, not shear yielding, should control the block shear tearing mode; therefore, use Equation 10.4-2. For Case (b), block shear cannot occur until the tension area ruptures as given by Equation 10.4-1. If Equation 10.4-2 (shear rupture on the small area and yielding on the large tension area) is checked for Case (b), a smaller P_o will result. In fact, as the shear area gets smaller and approaches zero, the use of Equation 10.4-2 for Case (b) would give a block shear strength based totally on yielding of the gross tensile area. Block shear is a rupture or tearing phenomenon not a yielding limit state. Therefore, the proper equation to use is the one with the larger rupture term.

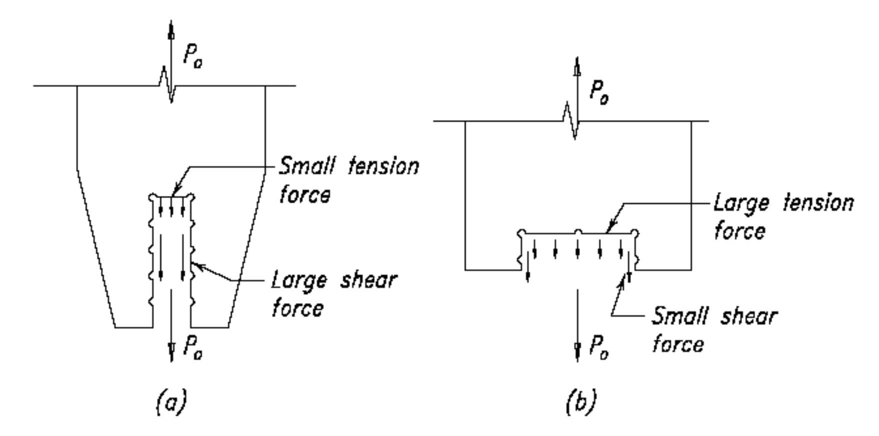


Figure. C10.4-2. Block shear rupture in tension.

SECTION C10.5
CONNECTING ELEMENTS

C10.5.2 **Design Strength of Connecting Elements in Tension.** Tests have shown that yield will occur on the gross section area before the tensile capacity of the net section is reached, if the ratio $A_n / A_g < 0.85$ (Kulak et al., 1987). Since the length of connecting elements is small compared to the member length, inelastic deformation of the gross section is limited. Hence, the effective net area A_n of the connecting element is limited to $0.85A_g$ in recognition of the limited inelastic deformation and to provide a reserve capacity.

SECTION C10.6
FILLERS

The practice of securing fillers by means of additional fasteners, so that they are, in effect, an integral part of a shear-connected component, is not required where a connection is designed to be a slip-critical connection using high-strength bolts. In such connections, the resistance to slip between filler and either connected part is comparable to that which would exist between the connected parts if no fill were present.

Filler plates may be used in lap joints of welded connections that splice parts of different thickness, or where there may be an offset in the joint.

SECTION C10.8
BEARING STRENGTH

The SBC 306 provisions for bearing on milled surfaces, Section 10.8, follow the same philosophy of ASD-based Specifications. In general, the design is governed by a deformation limit state at service loads resulting in stresses nominally at 9/10 of yield. Adequate safety is provided by post-yield strength as deformation increases. Tests on pin connections (Johnston, 1939) and on rockers (Wilson, 1934) have confirmed this behavior.

As used throughout the SBC 306, the terms "milled surface," "milled," and "milling" are intended to include surfaces which have been accurately sawed or finished to a true plane by any suitable means.

SECTION C10.9
COLUMN BASES AND BEARING ON CONCRETE

The equations for resistance of concrete in bearing are the same as SBC-304 except that this specification equations use $\phi = 0.60$ while SBC-304 uses $\phi = 0.70$, since SBC-304 specifies larger load factors than the ASCE load factors stipulated by this code requirement.

CHAPTER 11
CONCENTRATED FORCES, PONDING AND FATIGUE

SECTION C11.1
FLANGES AND WEBS WITH CONCENTRATED FORCES

C11.1.1 **Design Basis**. The SBC 306 separates flange and web strength requirements into distinct categories representing different limit state criteria, i.e., flange local bending (Section 11.1.2), web local yielding (Section 11.1.3), web crippling (Section 11.1.4), web sidesway buckling (Section 11.1.5), web compression buckling (Section 11.1.6), and web panel-zone shear (Section 11.1.7).

These criteria are applied to two distinct types of concentrated forces which act on member flanges. Single concentrated forces may be tensile, such as those delivered by tension hangers, or compressive, such as those delivered by bearing plates at beam interior positions, reactions at beam ends, and other bearing connections. Double concentrated forces, one tensile and one compressive, form a couple on the same side of the loaded member, such as that delivered to column flanges through welded and bolted moment connections. See Carter (1999) for guidelines on column stiffener design.

C11.1.2 **Flange Local Bending**. Where a tensile force is applied through a plate welded across a flange, that flange must be sufficiently rigid to prevent deformation of the flange and the corresponding high-stress concentration in the weld in line with the web.

The effective column flange length for local flange bending is $12t_f$ (Graham, et al., 1959). Thus, it is assumed that yield lines form in the flange at $6t_f$ in each direction from the point of the applied concentrated force. To develop the fixed edge consistent with the assumptions of this model, an additional $4t_f$ and therefore a total of $10t_f$, is required for the full flange-bending strength given by Equation 11.1-1. In the absence of applicable research, a 50 percent reduction has been introduced for cases wherein the applied concentrated force is less than $10t_f$ from the member end.

This criterion given by Equation 11.1-1 was originally developed for moment connections, but it also applies to single concentrated forces such as tension hangers consisting of a plate welded to the bottom flange of a beam and transverse to the beam web.

C11.1.3 **Web Local Yielding**. The web strength criteria have been established to limit the stress in the web of a member into which a force is being transmitted. It should matter little whether the member receiving the force is a beam or a column; however, Galambos (1976) and AISC (1978), references upon which the SBC 306 is based, did make such a distinction. For beams, a 2:1 stress gradient through the flange was used, whereas the gradient through column flanges was 2½:1. In Section 11.1.3, the 2½:1 gradient is used for both cases.

This criterion applies to both bearing and moment connections.

C11.1.4 **Web Crippling**. The expression for resistance to web crippling at a concentrated force is a departure from earlier specifications (IABSE, 1968; Bergfelt, 1971; Hoglund, 1971; and Elgaaly, 1983). Equations 11.1-4 and 11.1-5 are based on research by Roberts (1981). The increase in Equation 11.1-5b for $N/d > 0.2$ was developed after additional testing (Elgaaly and Salkar, 1991) to better represent the effect of longer bearing lengths at ends of members. All tests were conducted on bare steel beams without the expected beneficial contributions of any connection or floor attachments. Thus, the resulting criteria are considered conservative for such applications.

These equations were developed for bearing connections, but are also generally applicable to moment connections. However, for the rolled shapes listed in Part 1 of the LRFD Manual with F_y not greater than 345 MPa, the web crippling criterion will never control the design in a moment connection except for a W12 x 50 (W310 x 74) or W10 x 33 (W250 x 49.1) column.

The web crippling phenomenon has been observed to occur in the web adjacent to the loaded flange. For this reason, a half-depth stiffener (or stiffeners) or a half-depth doubler plate is expected to eliminate this limit state.

C11.1.5 **Web Sidesway Buckling**. The web sidesway buckling criterion was developed after observing several unexpected failures in tested beams (Summers and Yura, 1982). In those tests the compression flanges were braced at the concentrated load, the web was squeezed into compression, and the tension flange buckled (see Figure C11.1-1).

Web sidesway buckling will not occur in the following cases. For flanges restrained against rotation:

$$\frac{h/t_w}{l/b_f} > 2.3 \qquad \text{(C11.1-1)}$$

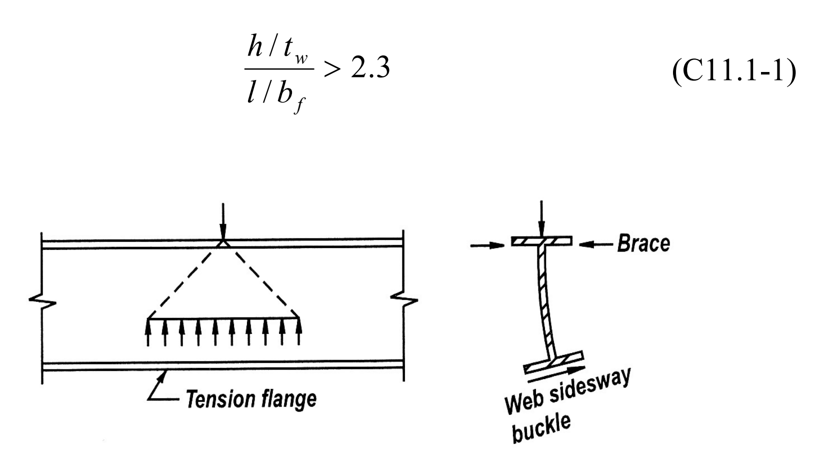


Figure. C11.1-1. Web sidesway buckling.

For flanges not restrained against rotation:

$$\frac{h/t_w}{l/b_f} > 1.7 \qquad \text{(C11.1-2)}$$

where l is as shown in Figure C11.1-2.

Web sidesway buckling can also be prevented by the proper design of lateral bracing or stiffeners at the load point. It is suggested that local bracing at both flanges be designed for one percent of the concentrated force applied at that point. Stiffeners must extend from the load point through at least one-half the beam or girder depth. In addition, the pair of stiffeners should be designed to carry the full load. If flange rotation is permitted at the loaded flange, neither stiffeners nor doubler plates will be effective.

In the 1st Edition LRFD Manual, the web sidesway buckling equations were based on the assumption that $h / t_f = 40$, a convenient assumption which is generally true for economy beams. This assumption has been removed so that the equations will be applicable to all sections.

This criterion was developed only for bearing connections and does not apply to moment connections.

C11.1.6 **Web Compression Buckling**. When compressive forces are applied to both flanges of a member at the same location, as by moment connections at both flanges of a column, the member web must have its slenderness ratio limited to avoid the possibility of buckling. This is done in the SBC 306 with Equation 11.1-8. This equation is applicable to a pair of moment connections, and to other pairs of compressive forces applied at both flanges of a member, for which N/d is small (<1). When N/d is not small, the member web should be designed as a compression member in accordance with Chapter 5.

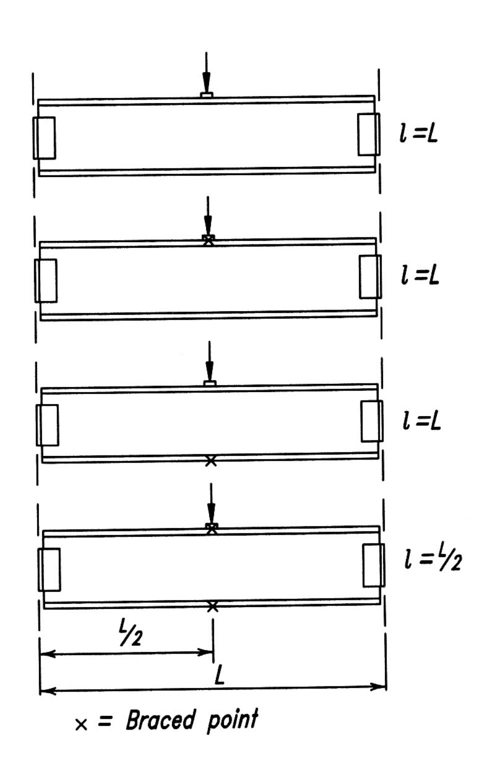


Figure. C11.1-2. Unbraced flange length.

Equation 11.1-8 is predicated on an interior member loading condition. In the absence of applicable research, a 50 percent reduction has been introduced for cases wherein the compressive forces are close to the member end.

Equation 11.1-8 has also traditionally been applied when there is a moment connection to only one flange of the column and compressive force is applied to only one flange. Its use in this case is conservative.

C11.1.7 **Web Panel-Zone Shear.** The column web shear stresses may be high within the boundaries of the rigid connection of two or more members whose webs lie in a common plane. Such webs should be reinforced when the calculated factored force ΣF_u along plane A-A in Figure C11.1-3 exceeds the column web design strength ϕR_v, where

$$\Sigma F_u = \frac{M_{u1}}{d_{m1}} + \frac{M_{u2}}{d_{m2}} - V_u \qquad \text{(C11.1-3)}$$

and

$M_{u1} = M_{u1L} + M_{u1G}$ = the sum of the moments due to the factored lateral load M_{u1L} and the moments due to factored gravity load M_{u1G} on the windward side of the connection, N-mm

$M_{u2} = M_{u2L} + M_{u2G}$ = the difference between the moments due to the factored lateral load M_{u2L} and the moments due to factored gravity load M_{u2G} on the windward side of the connection, N-mm

d_{m1}, d_{m2} = distance between flange forces in a moment connection, mm

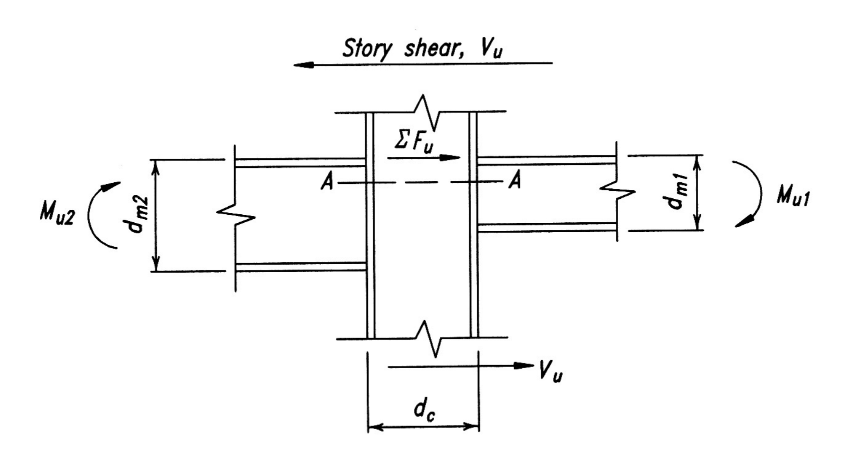


Figure. C11.1-3. Forces in panel zone.

Conservatively, 0.95 times the beam depth has been used for d_m in the past.

If $\Sigma F_u \leq \phi R_v$, no reinforcement is necessary, i.e., $t_{req} \leq t_w$, where t_w is the column web thickness.

Consistent with elastic first order analysis, Equations 11.1-9 and 11.1-10 limit panel-zone behavior to the elastic range. While such connection panels possess large reserve capacity beyond initial general shear yielding, the corresponding inelastic joint deformations may adversely affect the strength and stability of the frame or story (Fielding and Huang, 1971, and Fielding and Chen, 1973). Panel-zone shear yielding affects the overall frame stiffness and,

therefore, the ultimate-strength second-order effects may be significant. The shear/axial interaction expression of Equation 11.1-10, as shown in Figure C11.1-4, is chosen to ensure elastic panel behavior.

If adequate connection ductility is provided and the frame analysis considers the inelastic panel-zone deformations, then the additional inelastic shear strength is recognized in Equations 11.1-11 and 11.1-12 by the factor

$$\left[1+\frac{3b_{cf}t_{cf}^2}{d_b d_c t_w}\right]$$

This inelastic shear strength has been most often utilized for design of frames in high seismic zones and should be used when the panel zone is to be designed to match the strength of the members from which it is formed.

The shear/axial interaction expression incorporated in Equation 11.1-12 recognizes the observed fact that when the panel-zone web has completely yielded in shear, the axial column load is carried in the flanges.

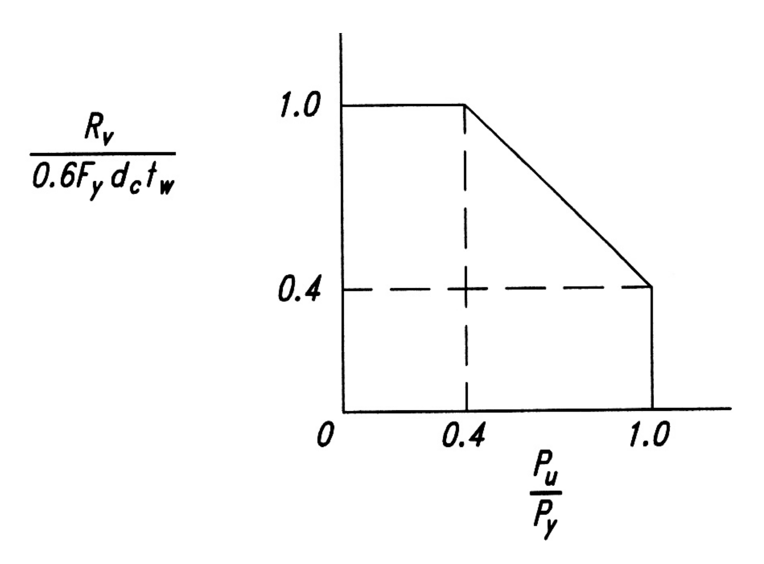


Figure. C11.1-4. Unbraced flange length.

SECTION C11.3
DESIGN FOR CYCLIC LOADING (FATIGUE)

In general, members or connections subject to less than a few thousand cycles of loading will not constitute a fatigue condition except possibly for cases involving full reversal of loading and particularly sensitive categories of details. This is because the admissible static design stress range will be limited by the admissible static design stress. At low levels of cyclic tensile stress, a point is reached where the stress range is so low that fatigue cracking will not initiate regardless of the number of cycles of loading. This level of stress is defined as the fatigue threshold, F_{TH}.

When fabrication details involving more than one category occur at the same location in a member, the stress range at that location must be limited to that of the most restrictive category. By locating notch-producing fabrication details in regions subject to a small range of stress, the need for a member larger than required by static loading will often be eliminated.

CHAPTER 12
SERVICEABILITY DESIGN CONSIDERATIONS

To satisfy the general design requirement for serviceability, the overall structure and the individual members, connections, and connectors shall be checked for serviceability.

Serviceability criteria are formulated to prevent disruptions of the functional use and damage to the structure during its normal everyday use. While malfunctions may not result in the collapse of a structure or in loss of life or injury, they can seriously impair the usefulness of the structure and lead to costly repairs. Neglect of serviceability may result in unacceptably flexible structures.

There are essentially three types of structural behavior which may impair serviceability:

(1) Excessive local damage (local yielding, buckling, slip, or cracking) that may require excessive maintenance or lead to corrosion.

(2) Excessive deflection or rotation that may affect the appearance, function, or drainage of the structure, or may cause damage to nonstructural components and their attachments.

(3) Excessive vibrations induced by wind or transient live loads which affect the comfort of occupants of the structure or the operation of mechanical equipment.

In allowable stress design, the Specification accounts for possible local damage with factors of safety included in the allowable stresses, while deflection and vibration are controlled, directly or indirectly, by limiting deflections and span-depth ratios. In the past, these rules have led to satisfactory performance of structures, with perhaps the exception of large open floor areas without partitions. In SBC 306 the serviceability checks should consider the appropriate loads, the response of the structure, and the reaction of the occupants to the structural response.

Examples of loads that may require consideration of serviceability include permanent live loads, wind, and earthquake; effects of human activities such as walking, dancing, etc.; temperature fluctuations; and vibrations induced by traffic near the building or by the operation of mechanical equipment within the building.

Serviceability checks are concerned with adequate performance under the appropriate load conditions. Elastic behavior can usually be assumed. However, some structural elements may have to be examined with respect to their long-term behavior under load.

It is difficult to specify limiting values of structural performance based on serviceability considerations because these depend to a great extent on the type of structure, its intended use, and subjective physiological reaction. For example, acceptable structural motion in a hospital clearly would be much less than in an ordinary industrial building. It should be noted that humans perceive levels of structural motion that are far less than motions that would cause any structural damage. Serviceability limits must be determined through careful consideration by the designer and client.

SECTION C12.1
CAMBER

The engineer should consider specifying camber when deflections at the appropriate load level present a serviceability problem.

SECTION C12.2
EXPANSION AND CONTRACTION

As in the case of deflections, the satisfactory control of expansion cannot be reduced to a few simple rules, but must depend largely upon the good judgment of qualified engineers.

The problem is more serious in buildings with masonry walls than with prefabricated units. Complete separation of the framing, at widely spaced expansion joints, is generally more satisfactory than more frequently located devices depend upon the sliding of parts in bearing, and usually less expensive than rocker or roller expansion bearings.

Creep and shrinkage of concrete and yielding of steel are among the causes, other than temperature, for dimensional changes.

SECTION C12.3
DEFLECTIONS, VIBRATION, AND DRIFT

C12.3.1 **Deflections.** Excessive transverse deflections or lateral drift may lead to permanent damage to building elements, separation of cladding, or loss of weather tightness, damaging transfer of load to non-load-supporting elements, disruption of operation of building service systems, objectionable changes in appearance of portions of the buildings, and discomfort of occupants.

The SBC 306 Specification does not provide specific limiting deflections for individual members or structural assemblies. Such limits would depend on the function of the structure. Provisions that limit deflections to a percentage of span may not be adequate for certain long-span floor systems; a limit on maximum deflection that is independent of span length may also be necessary to minimize the possibility of damage to adjoining or connecting nonstructural elements.

Deflection calculations for composite beams should include an allowance for slip for short-term deflection calculations, and for creep and shrinkage for long-term deflection calculations (see Commentary Section C9.3.2).

C12.3.2 **Floor Vibration**. The increasing use of high-strength materials and efficient structural schemes leads to longer spans and more flexible floor systems. Even though the use of a deflection limit related to span length generally precluded vibration problems in the past, some floor systems may require explicit consideration of the dynamic, as well as the static, characteristics of the floor system.

The dynamic response of structures or structural assemblies may be difficult to analyze because of difficulties in defining the actual mass, stiffness, and damping characteristics. Moreover, different load sources cause varying responses. For example, a steel beam-concrete slab floor system may respond to live

loading as a non-composite system, but to transient excitation from human activity as an orthotropic composite plate. Nonstructural partitions, cladding, and built-in furniture significantly increase the stiffness and damping of the structure and frequently eliminate potential vibration problems. The damping can also depend on the amplitude of excitation.

The general objective in minimizing problems associated with excessive structural motion is to limit accelerations, velocities, and displacements to levels that would not be disturbing to the building occupants. Generally, occupants of a building find sustained vibrations more objectionable than transient vibrations.

The levels of peak acceleration that people find annoying depend on frequency of response. Thresholds of annoyance for transient vibrations are somewhat higher and depend on the amount of damping in the floor system. These levels depend on the individual and the activity at the time of excitation.

The most effective way to reduce effects of continuous vibrations is through vibration isolation devices. Care should be taken to avoid resonance, where the frequency of steady-state excitation is close to the fundamental frequency of the system. Transient vibrations are reduced most effectively by increasing the damping in the structural assembly. Mechanical equipment which can produce objectionable vibrations in any portion of a structure should be adequately isolated to reduce the transmission of such vibrations to critical elements of the structure.

C12.3.3 **Drift.** The SBC 306 does not provide specific limiting values for lateral drift. If a drift analysis is desired, the stiffening effect of non-load-supporting elements such as partitions and in filled walls may be included in the analysis of drift. Some irrecoverable inelastic deformations may occur at given load levels in certain types of construction. The effect of such deformations may be negligible or serious, depending on the function of the structure, and should be considered by the designer on a case by case basis.

The deformation limits should apply to structural assemblies as a whole. Reasonable tolerance should also be provided for creep. Where load cycling occurs, consideration should be given to the possibility of increases in residual deformation that may lead to incremental failure.

SECTION C12.5
CORROSION

Steel members may deteriorate in particular service environments. This deterioration may appear either in external corrosion, which would be visible upon inspection, or in undetected changes that would reduce its strength. The designer should recognize these problems by either factoring a specific amount of damage tolerance into the design or providing adequate protection systems (e.g., coatings, cathodic protection) and/or planned maintenance programs so that such problems do not occur.

CHAPTER 13
FABRICATION, ERECTION AND QUALITY CONTROL

SECTION C13.2
FABRICATION

C13.2.1 **Cambering, Curving, and Straightening.** The use of heat for straightening or cambering members is permitted for ASTM A514/A514M and ASTM A852/A852M steel, as it is for other steels. However, the maximum temperature permitted is 593°C compared to 649°C for other steels.

Cambering of flexural members, when required by the contract documents, may be accomplished in various ways. In the case of trusses and girders, the desired curvature can be built in during assembly of the component parts. Within limits, rolled beams can be cold-cambered at the producing mills.

Local application of heat has come into common use as a means of straightening or cambering beams and girders. The method depends upon an ultimate shortening of the heat-affected zones. A number of such zones, on the side of the member that would be subject to compression during cold-cambering or "gagging," are heated enough to be "upset" by the restraint provided by surrounding unheated areas. Shortening takes place upon cooling.

While the final curvature or camber can be controlled by these methods, it must be realized that some deviation, due to workmanship error and permanent change due to handling, is inevitable.

C13.2.2 **Thermal Cutting**. Preferably thermal cutting shall be done by machine. The requirement for a positive preheat of 66°C minimum when thermal cutting beam copes and weld access holes in ASTM A6/A6M Group 4 and 5 shapes, and in built-up shapes made of material more than 50 mm thick, tends to minimize the hard surface layer and the initiation of cracks.

C13.2.5 **Bolted Construction.** In the past, it has been required to tighten all ASTM A325 or A325M and A490 or A490M bolts in both slip-critical and bearing-type connections to a specified tension. The requirement was changed in 1985 to permit most bearing-type connections to be tightened to a snug-tight condition.

In a snug-tight bearing connection, the bolts cannot be subjected to tension loads, slip can be permitted and loosening or fatigues due to vibration or load fluctuations are not design considerations.

It is suggested that snug-tight bearing-type connections be used in applications where A307 bolts would be permitted.

This section provides rules for the use of oversized and slotted holes paralleling the provisions which have been in the RCSC Specification (RCSC, 1994) since 1972, extended to include A307 bolts which are outside the scope of the high-strength bolt specifications.

SECTION C13.3
SHOP PAINTING

The surface condition of steel framing disclosed by the demolition of long-standing buildings has been found to be unchanged from the time of its erection, except at isolated spots where leakage may have occurred. Even in the presence of leakage, the shop coat is found to be of minor influence.

The SBC 306 does not define the type of paint to be used when a shop coat is required. Conditions of exposure and individual preference with regard to finish paint are factors which bear on the selection of the proper primer. Hence, a single formulation would not suffice. For a comprehensive treatment of the subject, see SSPC (1989).

C13.3.5 **Surfaces Adjacent to Field Welds.** The SBC 306 allows for welding through surface materials, including appropriate shop coatings that do not adversely affect weld quality nor create objectionable fumes.

SECTION C13.4
ERECTION

C13.4.4 **Fit of Column Compression Joints and Base Plates.** Tests on spliced full-size columns with joints that had been intentionally milled out-of-square, relative to either strong or weak axis, demonstrated that the load-carrying capacity was the same as that for a similar unspliced column. In the tests, gaps of 2 mm were not shimmed; gaps of 6 mm were shimmed with non-tapered mild steel shims. Minimum size partial-joint-penetration welds were used in all tests. No tests were performed on specimens with gaps greater than 6 mm.

C13.4.5 **Field Welding.** The purpose of wire brushing shop paint on surfaces adjacent to joints to be field welded is to reduce the possibility of porosity and cracking and also to reduce any environmental hazard. Although there are limited tests which indicate that painted surfaces result in sound welds without wire brushing, other studies have resulted in excessive porosity and/or cracking when welding coated surfaces. Wire brushing to reduce the paint film thickness minimizes rejectable welds. Grinding or other procedures beyond wire brushing is not necessary.

CHAPTER 14
EVALUATION OF EXISTING STRUCTURES

SECTION C14.1
GENERAL PROVISIONS

The load combinations referred to in this chapter reflect gravity loading because it is the most prevalent condition encountered. If other loading conditions are a consideration, such as lateral loads, the appropriate load combination from SBC 301 or from the applicable building code should be used. The Engineer of Record for a project is generally established by the owner.

SECTION C14.2
MATERIALS PROPERTIES

C14.2.1 **Determination of Required Tests.** The extent of tests required depends on the nature of the project, the criticality of the structural system or member evaluated, and the availability of records pertinent to the project. Thus, the Engineer of Record is required to determine the specific tests required and the locations from which specimens are to be obtained.

C14.2.2 **Tensile Properties.** Samples required for tensile tests should be removed from regions of reduced stress, such as at flange tips at beam ends and external plate edges, to minimize the effects of the reduced area. The number of tests required will depend on whether they are conducted to merely confirm the strength of a known material or to establish the strength of some other steel. Guidance on the appropriate minimum number of tests is available (FEMA, 1997).

It should be recognized that the yield stress determined by standard ASTM methods and reported by mills and testing laboratories is somewhat greater than the static yield stress because of dynamic effects of testing. Also, the test specimen location may have an effect. These effects have already been accounted for in the nominal strength equations in the Specification. However, when strength evaluation is done by load testing, this effect should be accounted for in test planning because yielding will tend to occur earlier than otherwise anticipated. The static yield stress, F_{ys}, can be estimated from that determined by routine application of ASTM methods, F_y, by the following equation (Galambos, 1978 and 1998):

$$F_{ys} = R(F_y - 27) \qquad \text{(C14.2-1)}$$

where

F_{ys} = static yield stress (MPa)

F_y = reported yield stress (MPa)

R = 0.95 for tests taken from web specimens

R = 1.00 for tests taken from flange specimens

The R factor in Equation C14.2-1 accounts for the effect of the coupon location on the reported yield stress.

C14.2.4 **Base Metal Notch Toughness.** The Engineer of Record shall specify the location of samples. Samples shall be cored, flame cut, or saw cut. The Engineer

of Record will determine if remedial actions are required, such as the possible use of bolted splice plates.

C14.2.5 **Weld Metal.** Because connections typically have a greater reliability index than structural members, strength testing of weld metal is not usually necessary. However, field investigations have sometimes indicated that complete-joint-penetration welds, such as at beam-to-column connections, were not made properly. The specified provisions in Section 14.2.4 provide a means for judging the quality of such a weld. Where feasible, any samples removed should be obtained from compression splices rather than tension splices, because the effects of repairs to restore the sampled area are less critical.

C14.2.6 **Bolts and Rivets.** Because connections typically have a greater reliability index than structural members, removal and strength testing of fasteners is not usually necessary. However, strength testing of bolts is required where they cannot be properly identified otherwise. Because removal and testing of rivets is difficult, assuming the lowest rivet strength grade simplifies the investigation.

SECTION C14.3
EVALUATION BY STRUCTURAL ANALYSIS

C14.3.2 **Strength Evaluation.** Resistance factors reflect variations in determining strength of members and connections, such as uncertainty in theory and variations in material properties and dimensions. If an investigation of an existing structure indicates that there are variations in material properties or dimensions significantly greater than those anticipated in new construction, the Engineer of Record should consider the use of more conservative values.

SECTION C14.4
EVALUATION BY LOAD TESTS

C14.4.1 **Determination of Live Road Rating by Testing.** Generally, structures that can be designed according to the provisions of this Specification need no confirmation of calculated results by test. However, special situations may arise when it is desirable to confirm by tests the results of calculations. Minimal test procedures are provided to determine the live load rating of a structure. However, in no case is the live load rating determined by test to exceed that which can be calculated using the provisions of the Code. This is not intended to preclude testing to evaluate special conditions or configurations that are not adequately covered by this Code.

It is essential that the Engineer of Record take all necessary precautions to ensure that the structure does not fail catastrophically during testing. A careful assessment of structural conditions before testing is a fundamental requirement. This includes accurate measurement and characterization of the size and strength of members, connections, and details. Shoring and scaffolding should be used as required in the proximity of the test area to mitigate against unexpected circumstances. Deformations must be carefully monitored and structural conditions must be continually evaluated. In some cases it may be desirable to monitor strains as well.

The Engineer of Record must use judgment to determine when deflections are becoming excessive and terminate the tests at a safe level even if the desired loading has not been achieved. Incremental loading is specified so that deformations can be accurately monitored and the performance of the structure carefully observed. Load increments should be small enough initially so that the onset of significant yielding can be determined. The increment can be reduced as the level of inelastic behavior increases, and the behavior at this level carefully evaluated to determine when to safely terminate the test. Periodic unloading after the onset of inelastic behavior will help the Engineer of Record determine when to terminate the test to avoid excessive permanent deformation or catastrophic failure.

It must be recognized that the margin of safety at the maximum load level used in the test may be very small, depending on such factors as the original design, the purpose of the tests, and the condition of the structure. Thus, it is imperative that all appropriate safety measures be adopted. It is recommended that the maximum live load used for load tests be selected conservatively. It should be noted that experience in testing more than one bay of a structure is limited.

Criteria limiting increases in deformations for a period of one hour have been given to ensure that the structure is stable at the loads evaluated.

C14.4.2 **Serviceability Evaluation.** In certain cases serviceability criteria must be determined by load testing. It should be recognized that complete recovery (i.e., return to initial deflected shape) after removal of maximum load is unlikely because of phenomena such as local yielding, slip at the slab interface in composite construction, creep in concrete slabs, localized crushing or deformation at shear connections in slabs, slip in bolted connections, and effects of continuity. Because most structures exhibit some slack when load is first applied, it is appropriate to project the load-deformation curve back to zero load to determine the slack and exclude it from the recorded deformations. Where desirable, the applied load sequence can be repeated to demonstrate that the structure is essentially elastic under service loads and that the permanent set is not detrimental.

SECTION C14.5
EVALUATION REPORT

Extensive evaluation and load testing of existing structures is often performed when appropriate documentation no longer exists or when there is considerable disagreement about the condition of a structure. The resulting evaluation is only effective if well documented, particularly when load testing is involved. Furthermore, as time passes, various interpretations of the results can arise unless all parameters of the structural performance, including material properties, strength, and stiffness, are well documented.

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